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AN ECONOMIC ANALYSIS OF EARTHQUAKE DESIGN LEVELS.(U)
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TITLE: AN ECONOMIC ANALYSIS OF EARTHQUAKE DESIGN LEVELS

AUTHOR: J. M. Ferritto

DATE: July 1982

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NAVAL CIVIL ENGINEERING LABORATORY
PORT HUENEME, CALIFORNIA 93043

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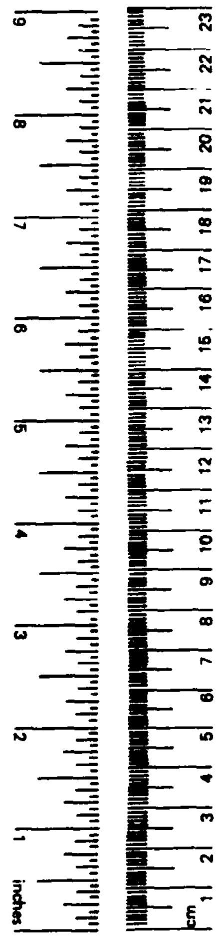
METRIC CONVERSION FACTORS

Approximate Conversions to Metric Measures

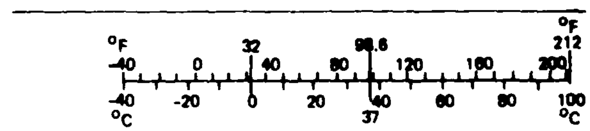
Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
in	inches	*2.5	centimeters	cm
ft	feet	30	centimeters	cm
yd	yards	0.9	meters	m
mi	miles	1.6	kilometers	km
		AREA		
in^2	square inches	6.5	square centimeters	cm^2
ft^2	square feet	0.09	square meters	m^2
yd^2	square yards	0.8	square meters	m^2
mi^2	square miles	2.6	square kilometers	km^2
	acres	0.4	hectares	ha
		MASS (weight)		
oz	ounces	28	grams	g
lb	pounds	0.45	kilograms	kg
	short tons (2,000 lb)	0.9	tonnes	t
		VOLUME		
tsp	teaspoons	5	milliliters	ml
Tbsp	tablespoons	15	milliliters	ml
fl oz	fluid ounces	30	milliliters	ml
c	cups	0.24	liters	l
pt	pints	0.47	liters	l
qt	quarts	0.95	liters	l
gal	gallons	3.8	liters	l
ft^3	cubic feet	0.03	cubic meters	m^3
yd^3	cubic yards	0.76	cubic meters	m^3
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C

*1 in = 2.54 (exactly). For other exact conversions and more detailed tables, see NBS Misc. Publ. 286, Units of Weights and Measures, Price $2.25, SD Catalog No. C13.10:286.



Approximate Conversions from Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
mm	millimeters	0.04	inches	in
cm	centimeters	0.4	inches	in
m	meters	3.3	feet	ft
m	meters	1.1	yards	yd
km	kilometers	0.6	miles	mi
		AREA		
cm^2	square centimeters	0.16	square inches	in^2
m^2	square meters	1.2	square yards	yd^2
km^2	square kilometers	0.4	square miles	mi^2
ha	hectares (10,000 m^2)	2.5	acres	
		MASS (weight)		
g	grams	0.035	ounces	oz
kg	kilograms	2.2	pounds	lb
t	tonnes (1,000 kg)	1.1	short tons	
		VOLUME		
ml	milliliters	0.03	fluid ounces	fl oz
l	liters	2.1	pints	pt
l	liters	1.06	quarts	qt
l	liters	0.26	gallons	gal
m^3	cubic meters	35	cubic feet	ft^3
m^3	cubic meters	1.3	cubic yards	yd^3
		TEMPERATURE (exact)		
°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	°F



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This report presents data on the cost of increasing the seismic design strength of buildings for three strengthening concepts: moment frame, braced frame, and shear wall. Damage is related to drift and acceleration of key elements of the structure. A damage matrix was constructed relating damage to design level and applied loading. An economic analysis was performed evaluating cost of strengthening, the present worth of expected damage, and the probability of site acceleration levels.

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EXECUTIVE SUMMARY

PRESENT PRACTICE

Present Navy seismic design policy recommends designing important new structures for an acceleration level equal to the site acceleration having an 80% chance of not being exceeded in 50 years. This is equivalent to a 225-year return time acceleration. Present policy, however, does not define what level of structural response should occur - should the structure remain elastic or should there be some amount of inelastic behavior?

THE PROBLEM

Design for the 225-year site acceleration often results in high seismic design levels which are difficult to achieve and are costly. The problem with the present Navy practice is that it does not clearly define a level of performance and safety of the structure because both the load, which has been defined, and the allowable deformation, which has not been defined, are necessary. Yet, despite the high design levels, a standardized risk or safety level is not being achieved. A complete specification of load and response is required so that a design procedure can be standardized. Further, it is necessary to distinguish the categories of important Navy structures that are mission-essential and cannot tolerate an interruption in operation from those which, because of high occupancy or other important function, warrant increased expenditure to limit damage. Seismic design levels must be analyzed with consideration given to the expected damage from a site's seismic activity and the costs of seismic strengthening.

PRESENT WORK EFFORT

FY81: An automated procedure was developed for conducting site seismicity using the historical epicenter data base and available geologic data. This procedure permits definition of the site acceleration probability distribution, quantifying the seismic exposure.

This study has reviewed cost increases for construction strengthening and expected damage from seismic shaking. The specification of a 225-year return-time acceleration does not produce optimal least total cost designs over all ranges of acceleration; rather, the least total cost design acceleration varies with site activity. It is not economically advantageous to design against high ground acceleration. The economic analysis procedures specified in NAVFAC P442 for the cases studied suggest the present worth of future damage is low enough that an earthquake design return time lower than that presently used (for a ductility of 1.0) is more efficient.

In evaluating seismic planning decisions, it was recommended that the Navy should:

1. Adopt a risk pooling policy, that provides a direct linear relation between expected losses and cost of strengthening.

2. Consider as acceptable investments only those instances where the benefits exceed the costs.

3. Limit critical buildings to Navy mission-essential structures of strategic value which must function after an earthquake. The remainder, such as hospitals, barracks, and aircraft hangars, should be analyzed in terms of the economics of strengthening and replacing damaged contents of the structure.

The work accomplished in FY81 is reported in NCEL Technical Report R-885.

FY82: To further develop the seismic economic analysis techniques and obtain more accurate data, a structure typical of Navy construction was selected for a detailed study. The structure was a three-story steel frame building built on the east coast for which detailed cost data and plans were available. Three methods of seismic strengthening were considered: moment frame, braced frame, and frame/concrete shear wall. Designs of each method were prepared for six levels of loading from 0.1g to 0.35g. Designs were set at a ductility of 1.0, which (1) allows for elastic analysis techniques to be utilized, (2) permits response to be related to other ductilities, and (3) offers the engineer the simplest dynamic analysis technique. Each design was then analyzed for a series of seismic levels and damage at each level evaluated. An economic analysis was then performed using the cost data, damage data, and seismic probability distributions from five sites. Results are discussed in the text.

In order to consider implementation of this work, it is recommended that the following definitions be adopted to clarify Navy requirements:

1. Mission Essential. Those facilities relating to the defense posture of the Navy mission which must function, without consideration of cost.

2. Very Important Structures. Those facilities warranting special attention to minimize damage and reduce the risk of loss of life and for which additional expenditure of funds is justified.

3. Important Structures. Structures which require designs in excess of code provisions but do not have the same level of importance as above. Optimal level of expenditure of funds is considered, minimizing the expected costs, damage, and strengthening.

Figure 14 summarizes, for the structure studied, design acceleration in terms of the site 225-year return-time acceleration for the moment frame, braced frame, and shear wall construction. Also shown is the present NAVFAC policy. It is suggested that Figure 14 is best suited for Important buildings similar to the structure studied. A penalty factor of five is suggested for use to weigh damage for Very Important facilities. Figure 15 presents design data for Very Important facilities similar to the structure studied. For true Mission-Essential facilities, procedures are given to identify required levels of performance and procedures to limit damage. The analysis of Mission-Essential facilities is deterministic in nature, providing for continuing operations under the maximum credible event.

OPTIONS FOR IMPLEMENTATION

For the classes of Important and Very Important structures, several options are available for implementation, depending on the level of effort that NAVFAC feels warranted. For all the options, the ductility performance level of 1.0 is used.

Option 1 - Limited Economic Analysis: The first option is to employ a limited economic analysis of the type performed here for each major construction project. Damage function of the type shown in Figures 4, 5, and 6, rather than a full damage matrix approach, should be sufficient to obtain meaningful data. Utilization of a damage function reduces the extent of analytical calculation required. The damage function should be representative of the class of structure being studied, with consideration of the type of strengthening system to be used. The economic analysis can be performed in conjunction with site seismicity studies. Appropriate software has been partially developed as part of this task. Development into a production code and documentation remain to be accomplished.

Option 2 - Prescribed Acceleration Reduction Levels: Figure 14 may be utilized as the basis for Important structures. In this option 225-year return-time acceleration is reduced, depending on the type of construction. These data are limited to the class of structure analyzed; however, it may be applicable to other classes also. Figure 15 may be utilized for Very Important structures to which special emphasis for reduced risk is deemed desirable. This work may be expanded to cover additional structures or structural materials to produce additional guidance.

Option 3 - Prescribed Acceleration Return Times: Design return times such as the following may be selected:

Important structures	100 years
Very Important structures	250 years

This approach ignores the variability of site seismicity but reduces the complexity.

SIGNIFICANCE TO NAVY

The FY82 Navy MILCON budget for new construction is \$1.45 billion, of which \$411 million is for buildings in seismic zones 3 and 4. Construction of buildings in seismically active zones therefore constitutes 28% of the Navy MILCON budget. Of this amount, seismic strengthening using present procedures costs on the order of \$16 million annually. It is estimated that implementation of a standardized design procedure will result in a more efficient utilization of funds, reduce risk, quantify expected losses, and possibly reduce costs by several million dollars annually.

CONTENTS

INTRODUCTION

The United States Navy has numerous bases located in active seismic regions, and each of these bases resembles a small city containing work areas and residential areas. With any seismic plan, establishing appropriate design levels which are safe, consistent with established knowledge, and economically effective must be considered. Because of the limited amount of available construction funds, an investigation of the economics of seismic strengthening is appropriate. What level of seismic design should be utilized considering costs of strengthening, the expected damage, and loss of life? This complex problem is the topic of this report.

A previous report (Ref 1) presents a detailed discussion of the state-of-the-art of seismic design and damage evaluation. This document will draw upon that information and develop an economic analysis of seismic design. Earlier results (Ref 1) using preliminary data indicated that designing for high levels of ground motion might not be cost-effective. To further explore this problem a typical structure was selected for detailed study. The structure chosen was representative of a class of structures utilized by the Navy for administration, light industrial work, or living quarters.

The structure selected was an actual three-story building for which detailed cost data and drawings were available. The building was recently constructed at an eastern Navy base in a nonseismic area. Thus, the nonseismic starting condition was established. The building was a steel frame structure, 185 by 185 feet in plan. Appendix A shows both the plan (Figure A-1) and elevated views of the building (Figure A-2). The latter also indicates the framing of the building

SEISMIC DESIGN

The selected structure was redesigned considering the structure to be new construction and being located in seismically active areas. Seismic design concepts were typical of conventional West Coast standard engineering design practice (Appendix B).

The structure was designed for six levels of peak ground acceleration: 0.10g to 0.35g with 5g increments. Elastic design spectra utilizing Newmark standard spectral shapes were utilized. Three concepts of seismic strengthening were utilized: (1) moment frame, (2) braced frame, and (3) shear wall. The performance level of the structure under the specified spectra was required to have a ductility equal to 1.0, such that members were to be at yield. This performance level was specified for several reasons. First, specifying a ductility of 1.0 is the same as specifying a higher acceleration and some ductility greater than 1.0. Second, use of a ductility equal to 1.0 allows the structural design engineer use of all elastic computer codes without need for a nonlinear analysis; further, nonlinear spectral techniques need not be used.

The required designs for the three concepts of strengthening and six load levels were performed under contract with a firm having significant experience in seismic design. As part of that effort, the contractor was also tasked to provide detailed cost estimates for seismic strengthening by treating the structure as new construction, using the available cost data on the existing structure, and making adjustments for West Coast construction practice. Results of the designs are presented in Reference 2.

Cost of Seismic Strengthening

Detailed structural costs were estimated based on the results of the six design cases for the three concepts of strengthening. The cost of the existing exterior steel frame construction was deducted from the total building cost, and then each new seismic framing system was added to obtain a new total building cost. Concrete or masonry seismic shear wall configurations, when utilized, were assumed to replace the existing 6-inch concrete block. Foundation redesign was included. Costs were adjusted to 1981 costs in the Los Angeles area. Figure 1a shows the seismic strengthening concepts; Figure 1b shows the increase in cost for seismic strengthening. The details of the cost estimating are given in Reference 2. Figure 2 gives the first mode periods of the structure for the three strengthening concepts as a function of design level. The moment frame period shows greatest variation with design acceleration.

Damage Evaluations

Damage to structural frame members, shear walls, and other elements associated with displacement are influenced by the interstory drift. Other elements tied to the floors, such as equipment or contents, are influenced by floor acceleration. Reference 1 is a detailed study of previous work in damage evaluation and will not be repeated here.

To evaluate the damage expected to the structure, each of the six design levels for each of the three design concepts of strengthening was analyzed for a series of applied seismic load levels. Nonlinear finite element techniques were employed. The program DRAIN-TABS was utilized to perform the analysis. Figure 3 gives the damping used for the analysis based on engineering practice; damping increased with the ratio of applied load to design level. Drift and floor acceleration time history responses were computed in the analysis. Effective response levels were selected at 65% of peak values and used in the damage prediction. The value of 65% has been used in past studies to approximate effective peak ground acceleration. This value, based on engineering practice, is used to reduce the peak values to a level of repeated sustained loading.

The detailed cost estimate was utilized to identify key elements of the structure to which dollar values could be associated. Repair factors for damage were estimated. The key elements were divided into drift- or acceleration-sensitive components, and values of drift and acceleration were then related to damage for each element.

Tables 1 and 2 give the damage ratios for each key element. It should be noted that a value is included for contents and that utilization of repair multipliers can result in costs exceeding the total cost of the structure. This is reasonable since demolition and removal costs would be required for major repairs.

Use of Tables 1 and 2 in conjunction with the drift and acceleration values from the nonlinear analysis resulted in Tables 3, 4, and 5 which presents the damage matrices giving damage as a function of design level and applied loading. Appendix C presents a detailed discussion of the damage matrix approach. Included in the damage matrix is the damage to the structure and the contents using the noted repair factors.

Moment Frame. The response of the moment frame structure is in the constant velocity region of the spectra for all six design ranges. It is significant to note that as the structure is stiffened, displacement is reduced; however, acceleration is increased. Damage is dependent on both displacement and acceleration. Note also that for a given applied load level, each of the six design cases is at a different damping level, with the weakest structure being most heavily damped. In the low applied loading level the strong structures are lightly damped, responding elastically with higher floor accelerations. The weaker structures are more heavily damped, responding inelastically with lower floor accelerations. In this range, stiffer structures receive greater damage; this condition exists to about 0.5g for the range of structures studied. Over 0.5g, the stiffer structures exhibit lower damage, as might be expected. The use of a single time history event with its unique frequency content results in minor response variations. Any single time history has unique frequency gaps and high points. Since the period of the structure changes with strengthening, secondary interactions occur between the frequency high points and structure periods such that the response at a particular design level might be slightly reduced or amplified ov the response of an ideal time history without gaps and high points. Further, the six design cases are not exact multiples but rather depend on selection of available structural shapes. These factors induce very minor dispersion in the results. A clear conclusion, however, is that stiffening in the low applied acceleration region does not reduce total damage. Figure 4 shows a plot of damage ratio as a function of applied load to design level. The data illustrate the effects of variation in period of the structure on the response. The damage ratio is a complex function of period, damping, range of non-linear behavior, and the mix of total damage caused by drift and acceleration.

Braced Frame. The response of the braced frame structure is in the constant acceleration region of the spectra for all six design ranges. The structure in its basic configuration with bracing is a much stiffer structure than the corresponding moment frame, pushing the response from the constant velocity region to the acceleration region. The resulting floor accelerations produced by the applied loading are higher than those of the moment frame while story drifts are reduced. In the medium and low level applied loading range, damage decreases with stiffening; however, at high load levels the acceleration dominates, resulting in higher damage with stiffening. Again, note that damping varies with the ratio of applied to design load level. Note also that three of the designs utilized 2-bay bracing, and three of the designs utilized 3-bay bracing. Figure 5 shows a plot of damage ratio as a function of applied load to design level. Again, note the effect of period variation on response.

Shear Wall. The shear wall/frame structure was the stiffest of the three concepts studied. Damage was generally least with this structure; however, collapse did occur for the 0.1g design at 0.9g applied load. The brittle nature and sudden shear failure are illustrated by the 0.29 damage ratio at 0.8g loading and the collapse at 0.9g loading (Table 5). In general, because of the low period of the structure, floor acceleration resulting from amplification of base motion was least; and in high applied acceleration load levels, attenuation occurred. Figure 6 shows a plot of damage ratio as a function of applied loading.

Site Seismic Probability

An automated procedure has been developed at NCEL to perform a seismic analysis using available historic data and geologic data. The objective of the seismicity study was to determine the probability of occurrence of acceleration at the site. To do this, site coordinates and the study bounds are specified in terms of latitude and longitude. A regional study is first performed in which all of the historic epicenters are used with an attenuation relationship to compute site acceleration for all historic earthquakes. A regression analysis is performed to obtain regional recurrence coefficients, and a map of epicenters is plotted. The regional recurrence can be used to compute the probability of site acceleration for randomly located events in the study area. Such a condition is used when individual faults are not known well enough to be specified.

Where individual fault areas can be specified, individual subsets of the historic data are used in conjunction with geologic data to determine fault recurrence coefficients; these are used to compute the probability of site acceleration from individual fault sources. The total risk is determined for all faults specified. Confidence bounds are given on the site acceleration as a function of probability of not being exceeded. Results of five case studies were utilized in this work.

NAVY ECONOMIC ANALYSIS

Reference 3 specifies procedures for economic analyses of facilities. The principles of the analyses are:

1. Insure an optimum allocation of scarce resources.
2. Effectively consider alternatives and life-cycle funding implications.
3. Recognize that money has value over time expressed by an interest rate.

This analysis, thus, must include the consideration that earthquake strengthening is expressed as a current cost increase to protect against a future dollar loss. The real world is complicated by cost increases through inflation. This means that to repair or replace the damaged building some time in the future will cost more than today. The work in the previous sections expresses costs of strengthening and damage as a percentage of building value to maintain a common reference. That premise recognizes increased value of the building and increased costs of repair. In an economic sense this may be expressed as letting the discount rate (the value of return on investment) be equal to the inflation rate.

The government has placed a value on money in time. NAVFAC P442 (Ref 3) and DODINST 7041.3 specify the discount rate as 10%. NAVFAC P442 states:

> "The rationale for adopting the private-sector rate of return as the discount rate for analyzing Government investment proposals turns on the notion that Government investments are funded with money taken from the private sector (preponderantly via taxation), are made in the ultimate behalf of the private sector (i.e., the individuals comprising it), and thus bear an implicit rate of return comparable to that of projects undertaken in the private sector. In this interpretation, 10% measures the opportunity cost of investment capital foregone by the private sector."

The 10% rate is a differential rate in addition to inflation.

When the present worth of the annual expected damage is considered using a discount rate of 10%, the present worth estimate of the damage would effectively be reduced by a factor of about 5. To restate this, the earthquake could occur at any point during the life of the structure; the best estimate is to consider an equivalent series of annual expected losses. The assumed life of the structure is 50 years based on NAVFAC seismic design criteria.* The present worth of this accumulated loss series can be computed, and its value is about one-fifth of the total expected loss.

It is important to note that the discount rate specified for use is actually a differential rate of 10% over the rate of inflation. It is recognized that the future cost of the repair would increase with time. One could use the differential rate and not consider inflation, or one could consider the rate of inflation to project an increased repair cost and then discount that cost using a discount rate of 10% plus the inflation rate. The results for modest inflation rates are approximately the same. The differential cost approach has been used in this study.

Included in the economic analysis is a value for injury and loss of life. This is discussed in depth in Reference 1. The value of loss of a life used in the analysis is $300,000. As discussed later results were not sensitive to the value selected.

SITE STUDIES

Five sites** were examined in light of the cost and damage data presented earlier and the probability of site acceleration distributions. Figures 7, 8, and 9 give the results of the increased cost of strengthening

*NAVFAC P442 usually uses 25 years for performing economic analysis. Use of 50 years was selected to conform to seismic criteria in use.

**Bremerton, Wash.; Memphis, Tenn.; San Diego, Calif.; Port Hueneme, Calif.; and Long Beach, Calif.

for each structure type at various design levels. The moment frame is the most expensive strengthening system and demonstrates clearly a minimum cost. Costs for the braced frame and shear wall systems do not vary as significantly with design level.

Based on the probability distribution data from the five sites, Figure 10 indicates the least-cost design acceleration in terms of the 225-year return-time acceleration (80% probability of not being exceeded in 50 years).

SENSITIVITY OF RESULTS

To evaluate the sensitivity of the results to variations in cost, damage, and the value of life, a number of studies were run in which data were varied by 25%. Figures 11 and 12 illustrate typical results for the Port Hueneme site. Variations in the computational process essentially shifted results along the cost axis. The conclusions on cost-effective design levels are relatively insensitive to changes in cost, damage, and value of life.

TYPES OF FACILITIES

Categories of Seismic Safety

The following definitions regarding earthquake safety for structures have been adopted by the Naval Facilities Engineering Command (NAVFAC):

Life Safety. This is the lowest acceptable safety level and requires that the structures resist collapse when exposed to the design earthquake and that nonstructural items do not present a hazard to personnel.

Mission Essential. Structures in this category must remain functional after exposure to the design earthquake. The following structures qualify as mission essential:

(a) Power plants and electrical distribution systems
(b) Water tanks
(c) Medical facilities
(d) Fire stations
(e) Vital communication facilities
(f) Those designated by the major claimant

The "mission" is the overall Navy mission, not the local activity function. Where similar facilities exist at other geographical locations or where operations can be carried on expediently, a structure is not considered essential from the overall Navy standpoint. Also, if suspension of the activity can be tolerated during a repair or replacement period the structure is not considered mission essential.

Critical Facilities. This is a special category of structure* that houses dangerous or toxic substances whose release would be hazardous;

*Dams are covered under the Dam Safety Act and investigated under a separate program.

in general, this includes chemicals or explosives. Facilities in this category should provide for basic life safety but do not necessarily need to remain functional after the design earthquake. However, they should be adequate to insure that earthquake damage will not cause the release of hazardous substance.

These above definitions reflect the work of the Interagency Seismic Safety Committee and really do not reflect the Navy's needs clearly. They do not give any guidance on the meaning of a design earthquake. Should it be determined probabilistically or deterministically? How safe is enough? A review of the proposed mission essential facilities includes medical facilities. Does this humanitarian action truly reflect the Navy sea mission? The author would prefer that the term "mission essential" be strictly limited to those facilities which must survive to perform the defense mission of the Navy, such as the vital communication facility. Power distribution may appear to be mission essential but really is not when more cost-effective solutions such as "no break" electrical backup generators are usually standard items at any communication facility since base power outages from mechanical generating failures cannot be tolerated.

For this report, the author uses the following definitions:

Mission Essential. Those facilities relating to the defense posture of the Navy mission which must function without consideration of cost.

Very Important Structures. Those facilities which warrant special attention to minimize damage and reduce risk and life loss and for which additional expenditure of funds is warranted.

Important Structures. Structures which require designs in excess of code provisions but do not have the same level of importance as above. Optimal level of expenditure of funds is considered, minimizing the expected costs, damage, and strengthening.

Mission-Essential and Very Important Structures

A question might arise as to what design level should be utilized for mission essential structures. Again, the term "mission essential" in this context is restricted to mission-related requirements to support a vital Navy mission. "Mission essential" is not assigned to structures solely on the number of inhabitants, but rather on the unique function of the structure. Theaters, schools, and hospitals are not "mission-essential" structures — they are very important structures at best.

For a mission essential structure, what design level should be used to insure survival? The answer must be presented in terms of the expected performance and exposure. What constitutes survival? Ten percent damage should be acceptable for many structures; however, for electronic functions, equipment tolerances to acceleration may have to be specified. Survival for a critical structure must be specified in terms of operational limits that cannot be exceeded; e.g., electronics not having more than 0.2g, or structural, mechanical, and electrical systems having limits of 5% damage with no structural members yielding, or similar limits with a performance requirement having been specified, the response analysis may be performed. Relatively complete assurance can only be achieved by calculation of the maximum credible event. This defines the upper bound of seismic shaking physically possible and is not time-related; often this level is very high. Performance of the mission essential structure must be evaluated at lower shaking levels. Variations in earthquake frequency with amplitude of shaking significantly influence response. Critical limiting conditions may be reached at less than maximum shaking. Since the number of true mission-essential structures is very limited, each should be treated separately, using damage evaluation techniques developed in the Site Studies section. To insure survival with minimal disruption, having multiple facilities, each spaced several hundred miles apart should be considered; thus, a seismic event affecting one site would not affect the other.

For other structures, a second category should be adopted, that of the very important structure. This is a class of structure to which special attention should be paid and more money spent on improving the probability of reducing damage. The approach taken should utilize adjustment factors to subjectively evaluate the degree of enhancement.

As an example, a penalty factor of 5 may be applied to the damage. By this, the computed damage is multiplied by 5. The penalty factor of 5 is selected since it cancels the discounting effect. In essence, it presents the analysis results for the case where an economic return (10%) is not sought on the investment. This then minimizes cost of strengthening directly against expected damage. Other values may also be used. Figure 13 shows how the design acceleration levels shift for this damage level. This same figure may be viewed as computing the damage analysis without discounting the future damage over the life of the structure.

Demonstration

The following example cases of buildings with moment frame construction are presented to clarify usage of the procedure.

Example 1 - Important Building.

225-year site seismicity acceleration - 0.3g
Type of construction - Moment frame
Category - Important building

What is the minimum total cost design acceleration level?

Answer - Use Figure 10; 0.1g; $\mu = 1.0$.

Design for 0.1g for an elastic design (ductility = 1.0).

Example 2 - Very Important Building.

225-year site seismicity acceleration - 0.3g
Type of construction - Moment frame
Category - Very important building

What is an appropriate design acceleration level?

Judgment - apply factor of 5.0 to damage

Answer - Use Figure 13; 0.18g; $\mu = 1.0$.

Design for 0.18g for an elastic design (ductility = 1.0).

Example 3 - Mission-Essential Building.

225-year site seismicity acceleration - 0.3g
Type of construction - Moment frame
Category - Mission-essential building

What is the appropriate design acceleration level?

Answer - Take following steps:

1. Evaluate operational performance requirements.

2. Evaluate seismic behavior; determine maximum credible event.

3. Select performance limits, drift, displacement, acceleration, and other factors.

4. Develop drift and acceleration relationships similar to those in Tables 1 and 2.

5. Develop trial design; analyze for motion in step 2.

6. Evaluate damage.

7. Repeat step 5 to limit damage.

OPTIONS FOR IMPLEMENTATION

For important and very important structures, there are several options for implementation, depending on the level of effort that NAVFAC feels warranted. For all the options, recall that the ductility performance level of 1.0 is used.

Option 1 - Limited Economic Analysis

The first option is to employ a limited economic analysis of the type performed here for each major construction project. A damage function of the type shown in Figures 4, 5, and 6 should be sufficient rather than a full damage matrix approach. Utilization of a damage function reduces the extent of analytical calculation required. The damage function should be representative of the class of structure being studied, with the type of strengthening system considered. The economic analysis can be performed in conjunction with site seismicity studies. Appropriate software has been partially developed as part of this task. Development into a production code and documentation remain to be accomplished.

Option 2 - Prescribed Acceleration Reduction Levels

In this option, 225-year return time accelerations are reduced, depending on the type of construction. These data are limited to the class of structure analyzed, but the option may be applicable to other classes also. Figure 14 may be utilized as the basis for important structures. Figure 15 may be utilized for very important structures to which special emphasis for reduced risk is deemed desirable. This work may be expanded to cover additional structures or structural materials to produce additional guidance.

Option 3 - Prescribed Acceleration Return Times

Design return times such as the following may be selected:

Important structures	100 years
Very important structures	250 years

This approach ignores the variability of site seismicity but reduces the complexity.

Procedure for Estimating Optimal Design (Option 1)

The following gives a brief step-by-step procedure using Option 1 for computing design levels and expected loss from seismic shaking.

1. Determination of Site Acceleration Probability. The site data, including location of faults, maximum credible magnitudes, site location, and other factors, are used as input to NCEL-developed program RECUR. This program computes the total* probability of exceeding various levels of acceleration at the site. The results are given as a table of probability and acceleration level.

2. Spectra Level. A spectrum is created by use of site-matched records or by use of a standard shape site-independent technique. The spectrum is scaled to the nominal acceleration desired.

3. Cost Function. The cost of seismic strengthening must be estimated for a range of seismic levels.

4. Damage Function. The structural plans must be analyzed either using modal analysis techniques or nonlinear structural analysis procedures to evaluate drifts and accelerations with load. Programs exist to perform automated analysis. Damage as a function of design and load must be determined either as a damage function or as a damage matrix (see Appendix C). A damage function approach should be sufficient.

5. Economic Analysis. The expected damage and cost of strengthening must be evaluated using the site seismicity probability distribution. A program can be formulated to perform this analysis.

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*"Total" is used to indicate that the probability represents the effect of all the faults acting together and does not simply use the single worst or governing fault.

CONCLUSION

Seismic strengthening costs are seen to be dependent on the type of strengthening system utilized; damage is correlated both to drift and acceleration. Strengthening alone limits drift damage but increases acceleration damage. Damage to a structure is a complex mechanism influenced by damping level, degree of inelastic behavior, acceleration level as well as drift level, and spectral region of response. Economic design levels appear to be somewhat greater than those indicated by building codes; however, design for the full 225-year acceleration would not be cost-effective for all cases. The most cost-effective design acceleration is a function of construction type and site seismic exposure.

Acceleration produces a significant amount of damage, and special care should be taken to design ceilings and lights to withstand acceleration. Shaking produces overturning of equipment, which is a significant factor, accounting for most mechanical and electrical losses.

Since stiffening produces increased acceleration, consideration should be given to development and utilization of isolation techniques.

Several options are presented for NAVFAC consideration. Option 1 presents the most detailed approach and presents the most accurate solution. Option 2 recommends levels of motion less than the full 225-year return time. Option 3 specifies specific return times.

SIGNIFICANCE TO NAVY

Table 6 presents the FY82 Navy MILCON Building Construction Program showing construction in seismic zones 3 and 4. A breakdown of the types of facilities is shown in Table 7. Table 8 shows planned future MILCON levels. The FY82 MILCON total of $411 million for seismic zones 3 and 4 is 28% of the $1,450 million total MILCON. Seismic strengthening costs are a significant factor in construction expenditures. Implementation of a standardized design procedure will provide for a more efficient use of funds, reduction in risk, quantification of levels of safety, and possible reduction of costs of several million dollars annually.

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1. Civil Engineering Laboratory. Technical Report R-885: Procedure for conducting site seismicity studies at Naval shore facilities and an economic analysis of risk, by J. M. Ferritto. Port Hueneme, Calif., Feb 1981.

2. Martin and Saunders. Contract Report: Seismic redesign of a Navy building. Costa Mesa, Calif., Apr 1981. (Contract No. 68305-81-C-0007)

3. Naval Facilities Engineering Command. NAVFAC P442: Command economic analysis handbook (revised Oct 1975). Alexandria, Va., Oct 1975.

Table 1. Damage Ratios - Drift

Element	Cost ($)	Repair Miltiplier	Damage Ratios for Following Interstory Drift in./in.								
			0.001	0.005	0.010	0.020	0.030	0.040	0.070	0.100	0.140
1a. Rigid Frames	117,500[a]	2.0	0	0.01	0.02	0.05	0.10	0.25	0.35	0.50	1.00
b. Braced Frames	a	2.0	0	0.03	0.14	0.22	0.40	0.85	1.0	1.0	1.0
c. Shear Walls	a	2.0	0	0.05	0.30	0.30	0.60	0.85	1.0	1.0	1.0
2a. Nonseismic Structural Frame	625,500	1.5	0	0.005	0.01	0.02	0.10	0.30	1.0	1.0	1.0
3. Masonry	417,600	2.0	0	0.10	0.20	0.50	1.0	1.0	1.0	1.0	1.0
4. Windows and Frames	120,600	1.5	0	0.30	0.80	1.0	1.0	1.0	1.0	1.0	1.0
5. Partitions, Architectural Elements	276,200	1.25	0	0.10	0.30	1.0	1.0	1.0	1.0	1.0	1.0
6. Floor	301,200	1.5	0	0.01	0.04	0.12	0.20	0.35	0.80	1.0	1.0
7. Foundation	412,100	1.5	0	0.01	0.04	0.10	0.25	0.30	0.50	1.0	1.0
8. Building Equipment and Plumbing	731,600	1.25	0	0.02	0.07	0.15	0.35	0.45	0.80	1.0	1.0
9. Contents	500,000	1.0	0	0.02	0.07	0.15	0.35	0.45	0.80	1.0	1.0

[a]Varies with design.

Table 2. Damage Ratios - Acceleration

Element	Cost ($)	Repair Multiplier	Damage Ratios for Following Floor Acceleration				
			0.08g	0.18g	0.50g	1.2g	1.4g
1. Floor and Roof	301,200	1.5	0.01	0.02	0.10	0.50	1.0
2. Ceilings and Lights	288,500	1.25	0.01	0.10	0.60	0.95	1.0
3. Building Equipment and Plumbing	731,600	1.25	0.01	0.10	0.45	0.60	1.0
4. Elevators	57,000	1.5	0.01	0.10	0.50	0.70	1.0
5. Foundations (Slab on Grade, Site-work)	412,100	1.5	0.01	0.02	0.10	0.50	1.0
6. Contents	500,000	1.05	0.05	0.20	0.60	0.90	1.0

Table 3. Damage Ratio - Moment Frame

Applied Load (g)	Damage Ratios for Following Design Acceleration						
	0.00g	0.10g	0.15g	0.20g	0.25g	0.30g	0.35g
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.10	0.10	0.08	0.08	0.09	0.11	0.11	0.11
0.20	0.17	0.11	0.13	0.15	0.17	0.19	0.21
0.30	0.24	0.16	0.20	0.23	0.22	0.22	0.24
0.40	0.37	0.24	0.26	0.27	0.27	0.26	0.29
0.50	0.56	0.37	0.29	0.32	0.29	0.29	0.31
0.60	0.82	0.55	0.35	0.36	0.33	0.33	0.32
0.70	1.03	0.68	0.40	0.39	0.35	0.34	0.35
0.80	1.13	0.75	0.43	0.43	0.38	0.38	0.38
0.90	1.25	0.83	0.46	0.45	0.42	0.40	0.39
1.00	1.50	0.89	0.50	0.48	0.46	0.42	0.41

Table 4. Damage Ratio - Braced Frame

Applied Load (g)	Damage Ratios for Following Design Acceleration						
	0.00g	0.10g	0.15g	0.20g	0.25g	0.30g	0.35g
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.10	0.10	0.08	0.08	0.07	0.07	0.07	0.06
0.20	0.17	0.14	0.14	0.14	0.12	0.12	0.11
0.30	0.24	0.23	0.21	0.20	0.17	0.16	0.16
0.40	0.37	0.29	0.28	0.27	0.24	0.23	0.23
0.50	0.56	0.32	0.31	0.30	0.27	0.26	0.26
0.60	0.82	0.34	0.34	0.32	0.29	0.29	0.28
0.70	1.03	0.37	0.37	0.35	0.31	0.34	0.35
0.80	1.13	0.42	0.41	0.41	0.36	0.37	0.39
0.90	1.25	0.45	0.43	0.43	0.47	0.45	0.52
1.00	1.50	0.49	0.48	0.47	0.56	0.61	0.71

Table 5. Damage Ratio - Shear Wall

Applied Load (g)	Damage Ratios for Following Design Acceleration						
	0.00g	0.10g	0.15g	0.20g	0.25g	0.30g	0.35g
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.10	0.10	0.04	0.04	0.04	0.04	0.02	0.02
0.20	0.17	0.08	0.07	0.07	0.07	0.07	0.07
0.30	0.24	0.11	0.11	0.10	0.09	0.07	0.10
0.40	0.37	0.15	0.14	0.13	0.12	0.11	0.11
0.50	0.56	0.20	0.18	0.16	0.14	0.11	0.11
0.60	0.82	0.24	0.23	0.18	0.17	0.14	0.14
0.70	1.03	0.26	0.26	0.20	0.17	0.17	0.17
0.80	1.13	0.29	0.28	0.24	0.21	0.12	0.20
0.90	1.25	1.20	0.31	0.25	0.23	0.20	0.19
1.00	1.50	1.50	0.32	0.28	0.25	0.23	0.22

Table 6. Summary of FY82 Navy MILCON Building Construction Projects in Severe Earthquake Areas (Zones 3 and 4)

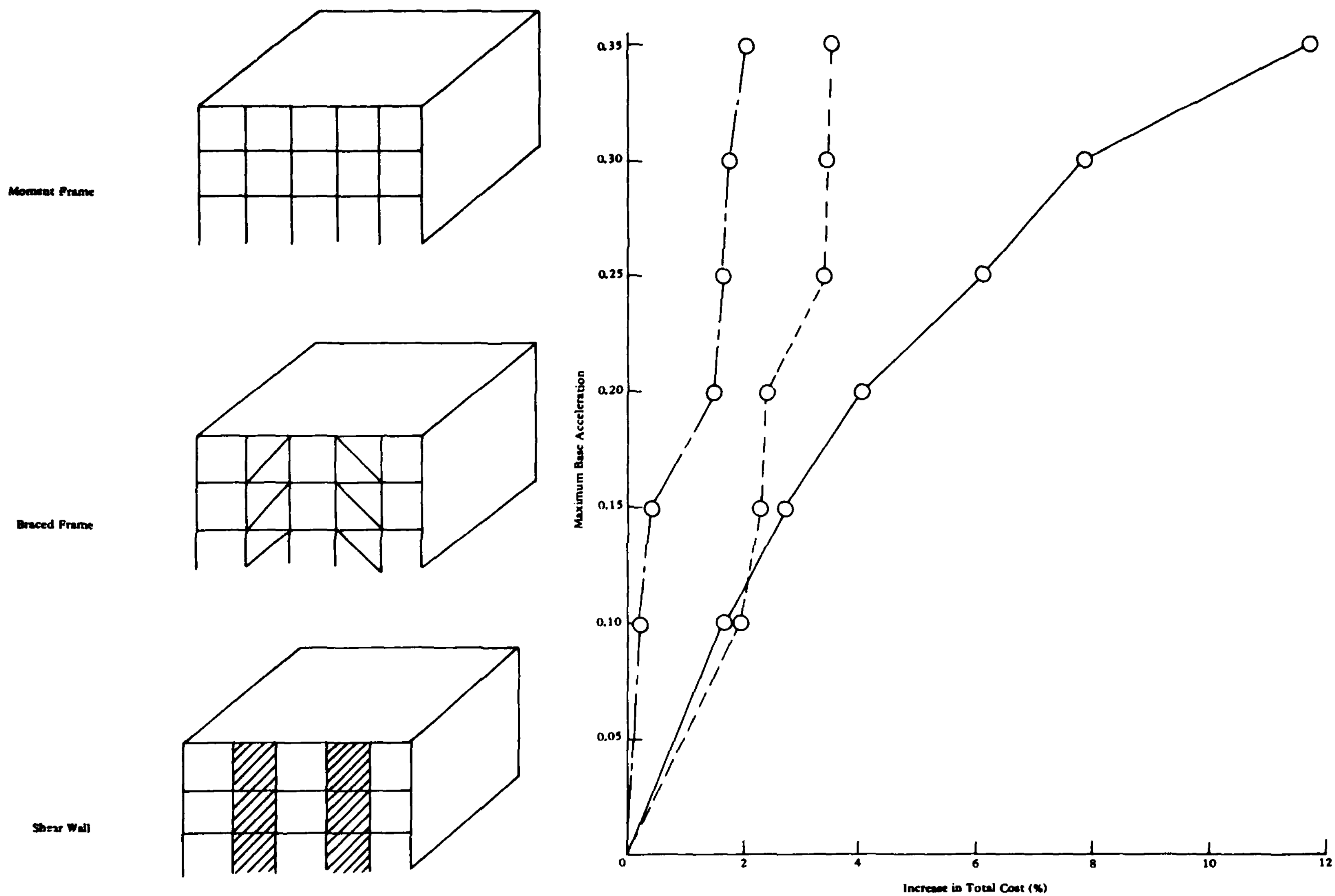
Area	Cost ($ in Thousands)	
	Individual	Total
Eastern United States		39,660
Boston/Portsmouth	2,900	
Charleston/Savannah	25,760	
St. Louis/Little Rock/ Memphis	11,000	
Western United States		352,060
California and Southwest Arizona	300,910	
Reno/Fallon/Hawthorne	12,000	
Seattle/Bremerton/Keyport-Bangor	32,600	
Alaska/Hawaii	6,550	
Pacific Ocean		20,200
Phillippine Islands	18,950	
Japan/Okinawa	1,250	
TOTAL		411,920

Table 7. Breakdown of Facilities in Seismic Zones 3 and 4

Facilities	Cost ($ in Thousands)
Housing	62,600
Warehouse and Miscellaneous Storage	14,550
Community	10,090
Administrative	24,200
Operational	68,480
Medical	204,000
Training and Other	28,000
TOTAL	411,920

Table 8. Planned MILCON Total Obligational Authority, Jan 1982

Fiscal Year	Cost ($ in Millions)
1982	1,450
1983	1,208
1984	1,616
1985	2,100
1986	2,289
1987	3,446



(a) Concepts.

(b) Increase in cost.

Figure 1. Seismic strengthening concepts and increase in cost for each type of strengthening.

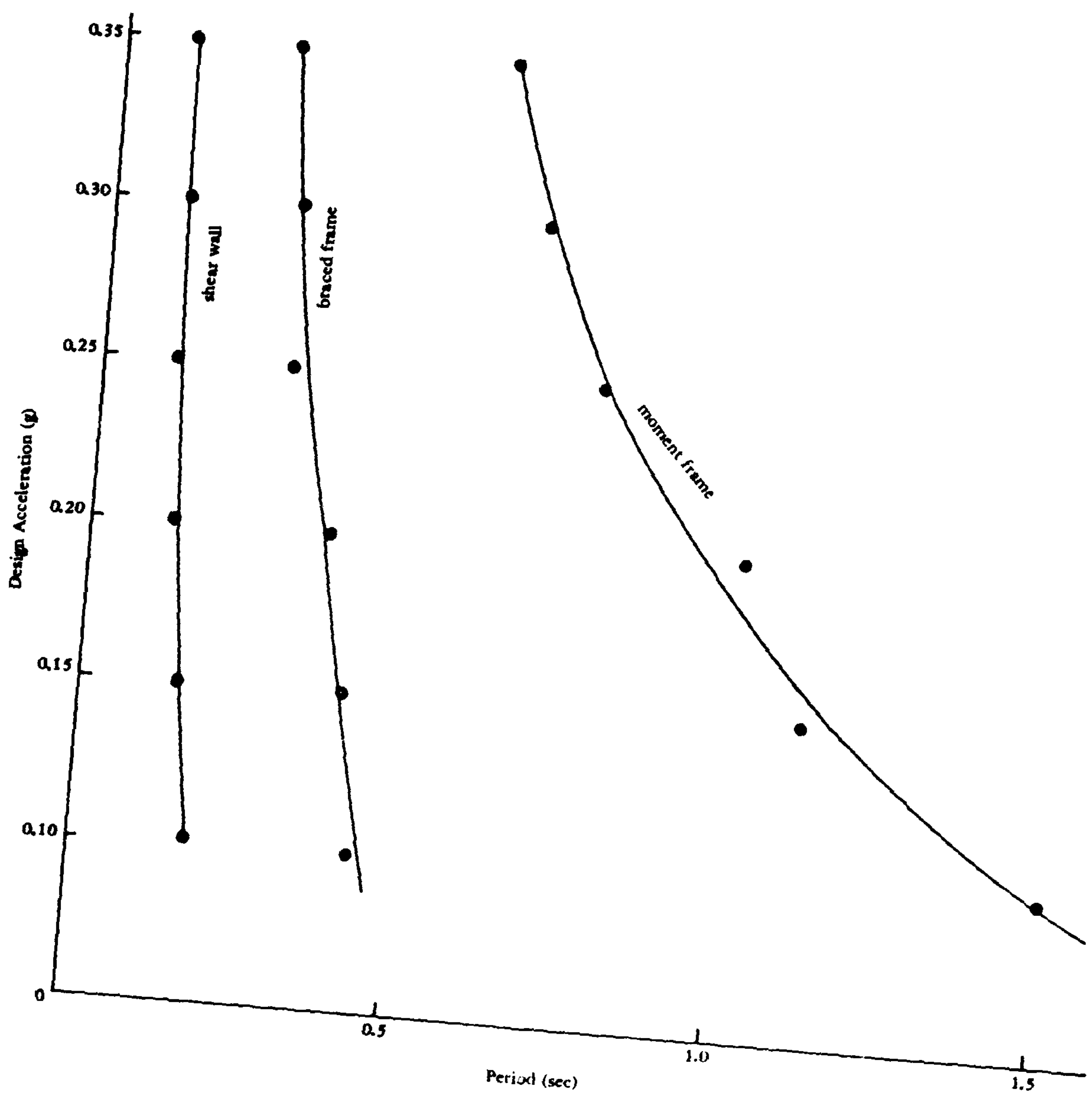


Figure 2. First mode period.

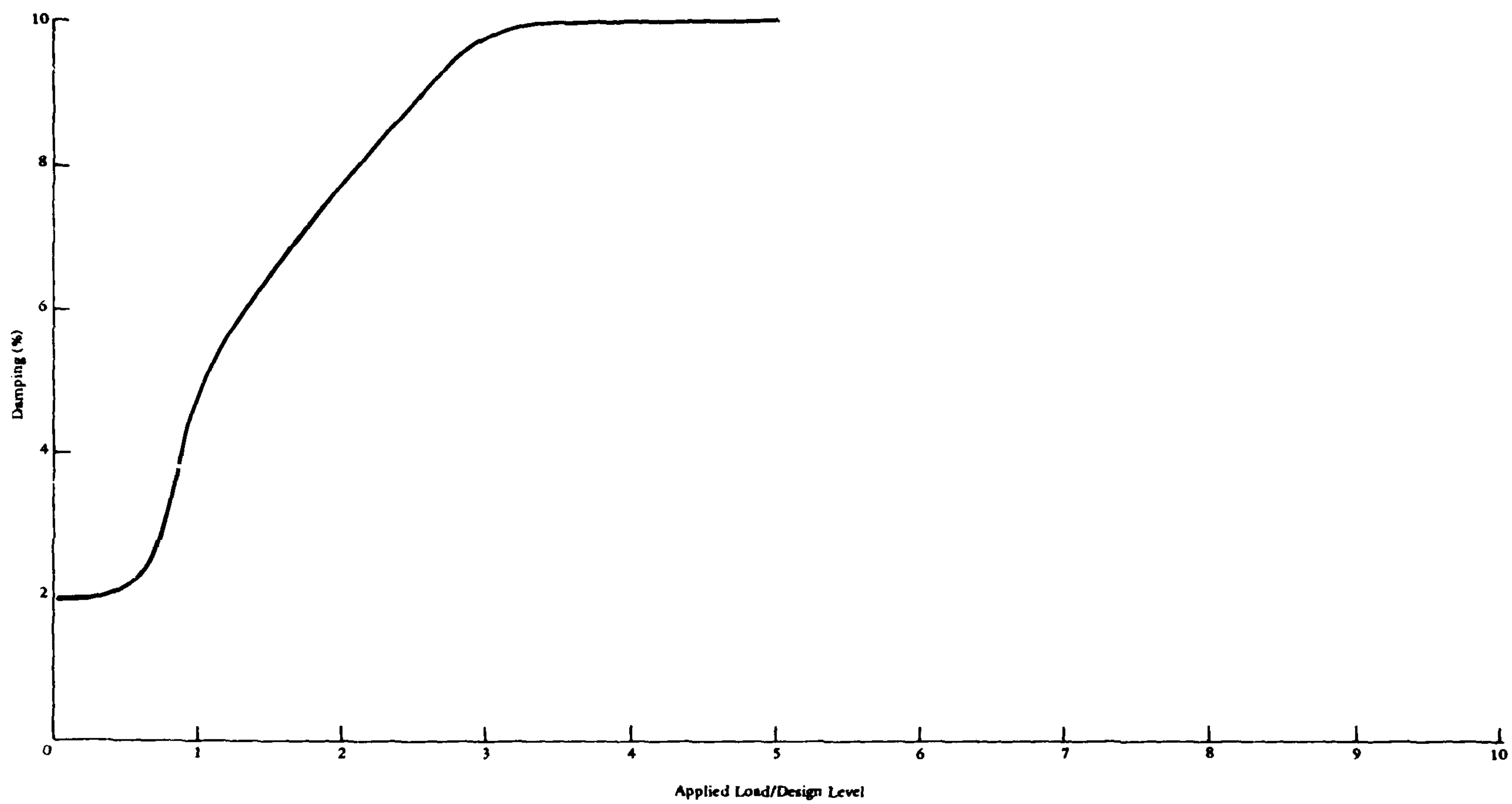


Figure 3. Damping.

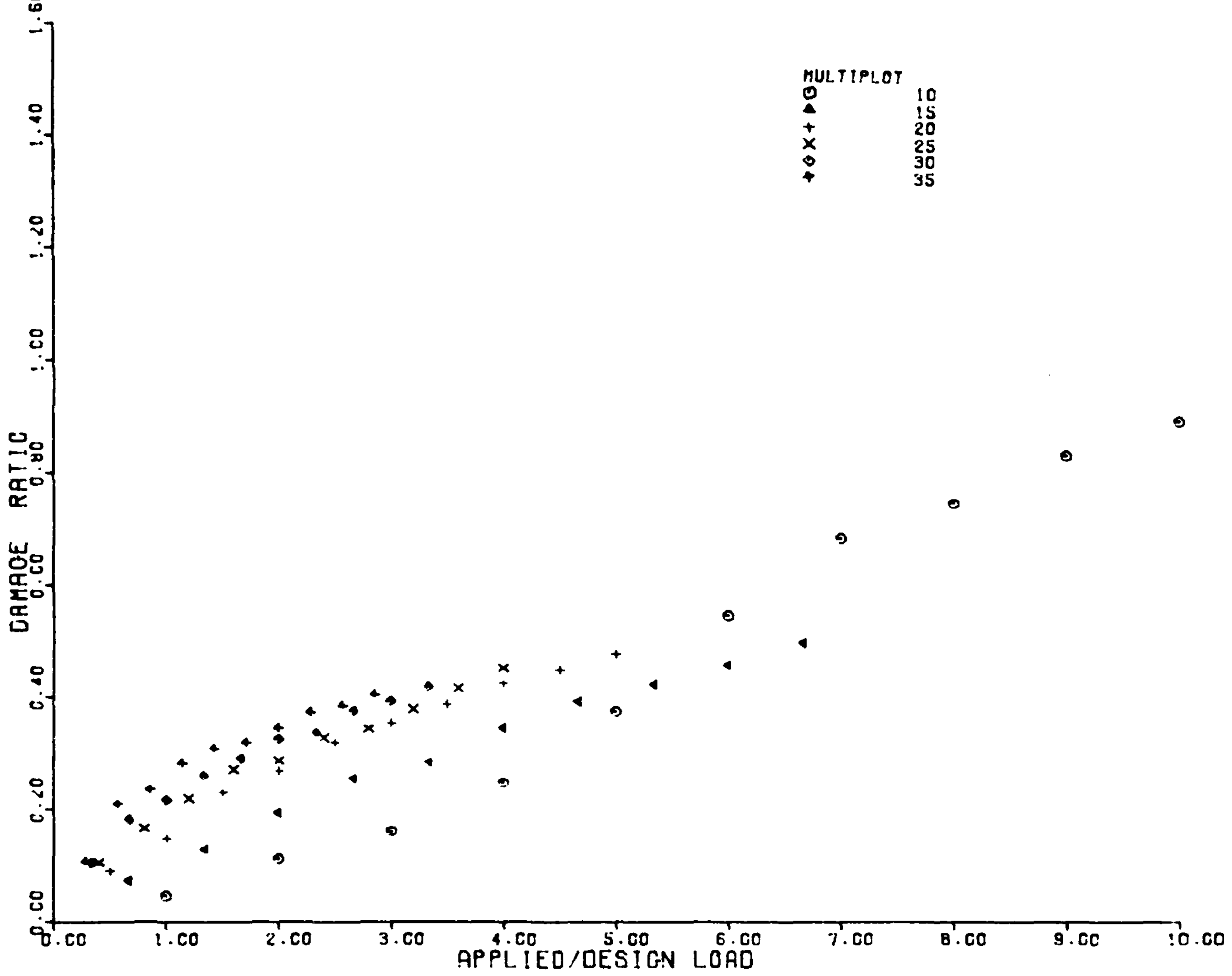


Figure 4. Damage ratio as a function of design and load, moment frame.

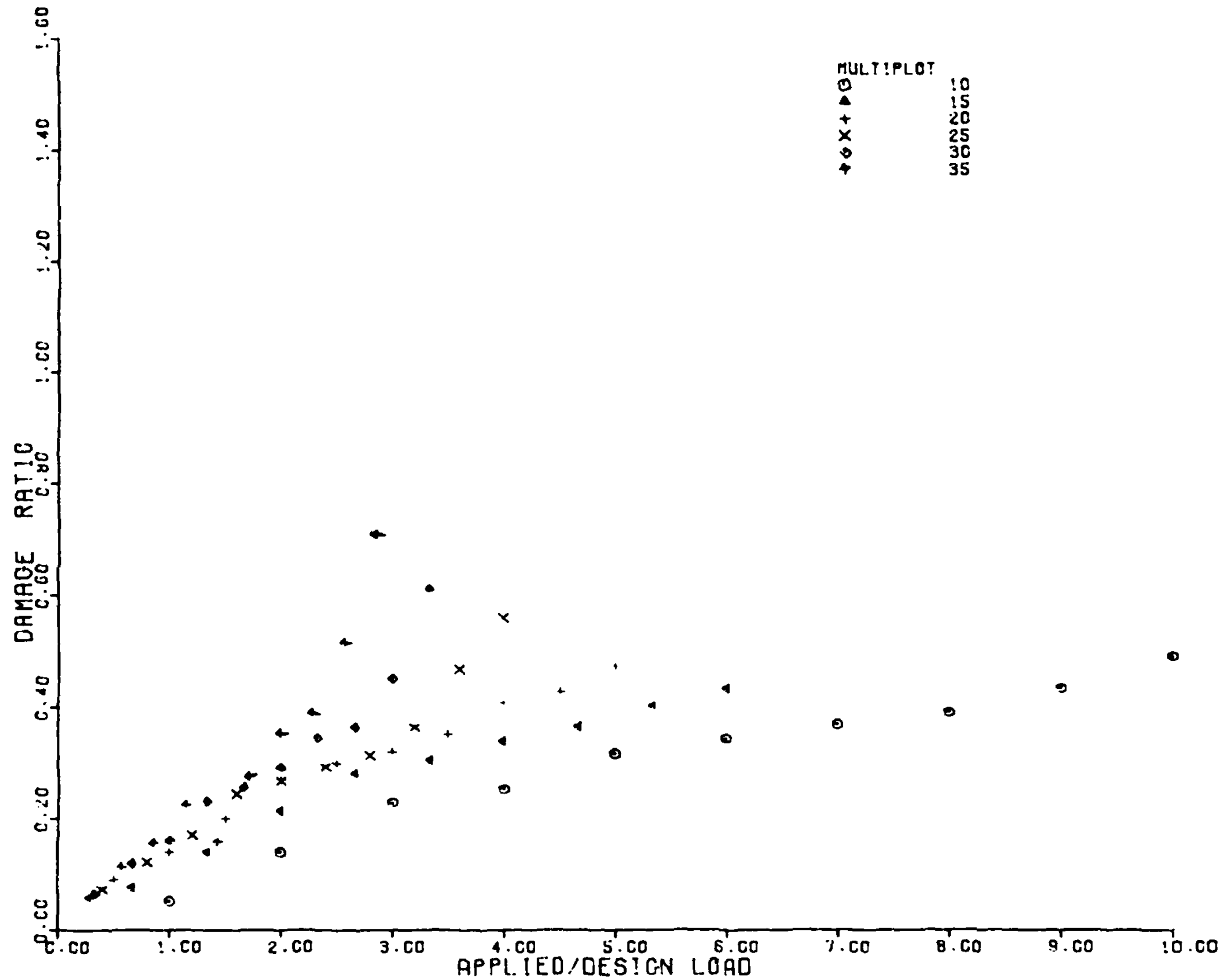


Figure 5. Damage ratio as a function of design and load, braced frame.

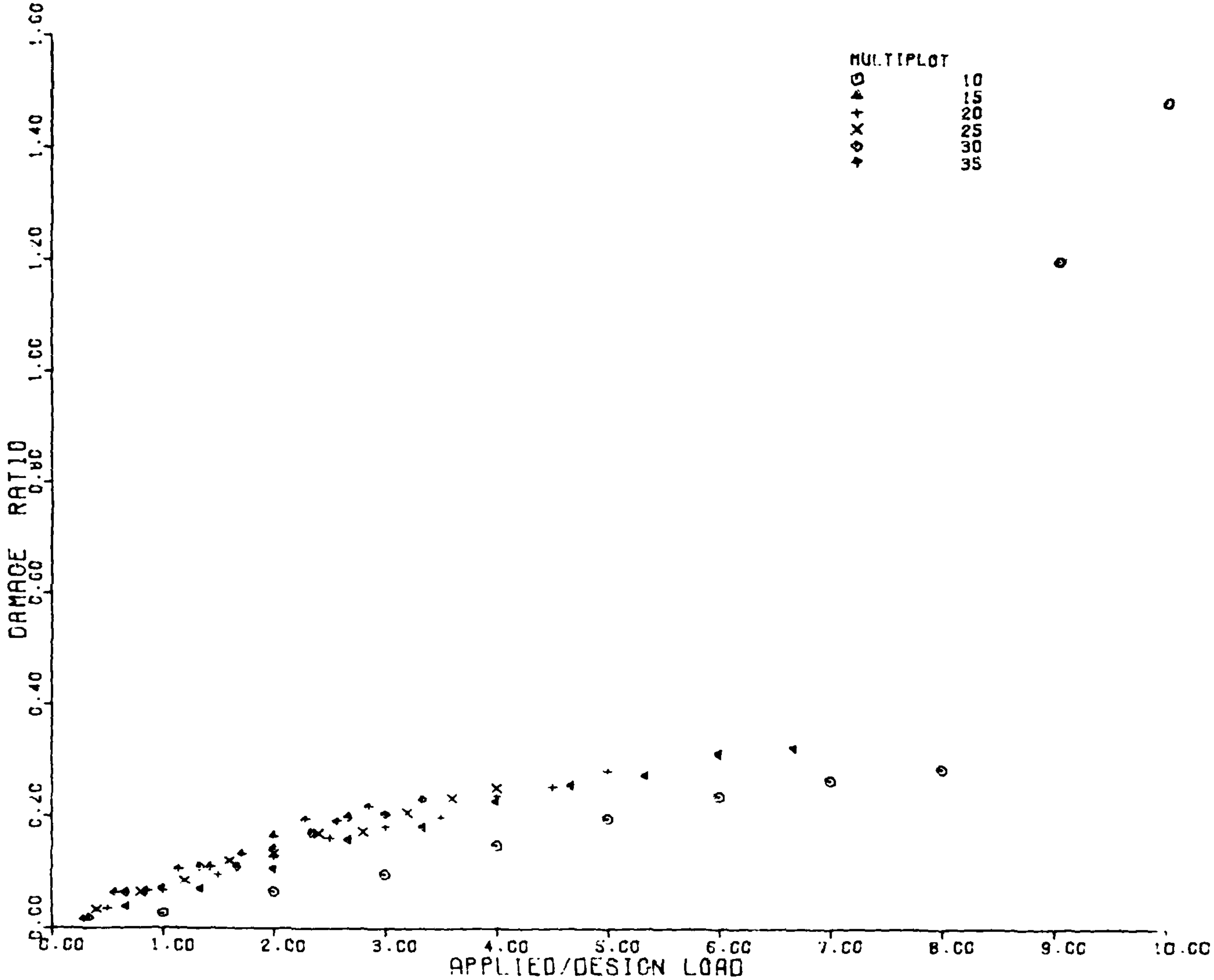


Figure 6. Damage ratio as a function of design and load, shear wall.

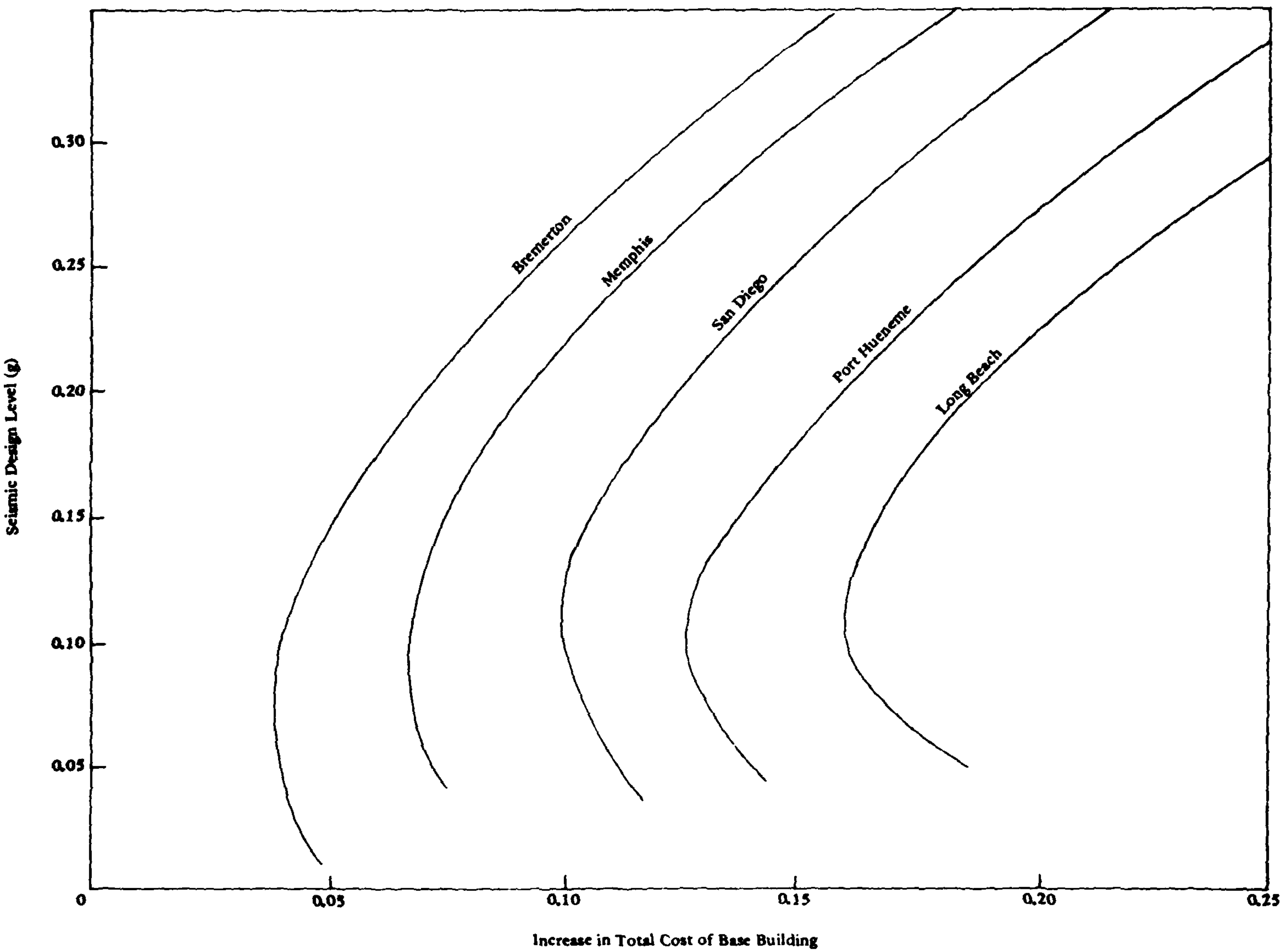


Figure 7. Increase in cost in various locations for moment frame structure.

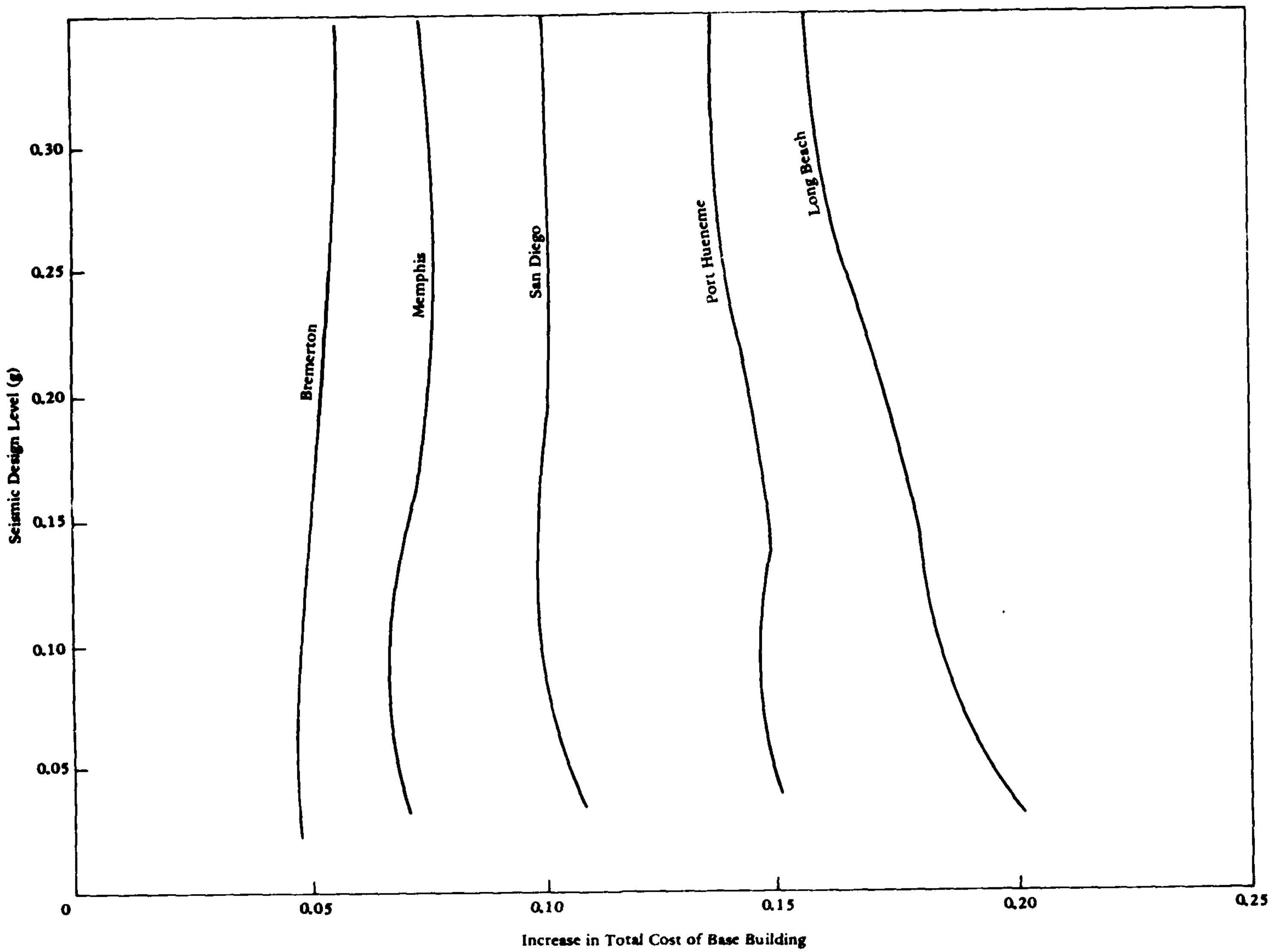


Figure 8. Increase in cost in various locations for braced frame structure.

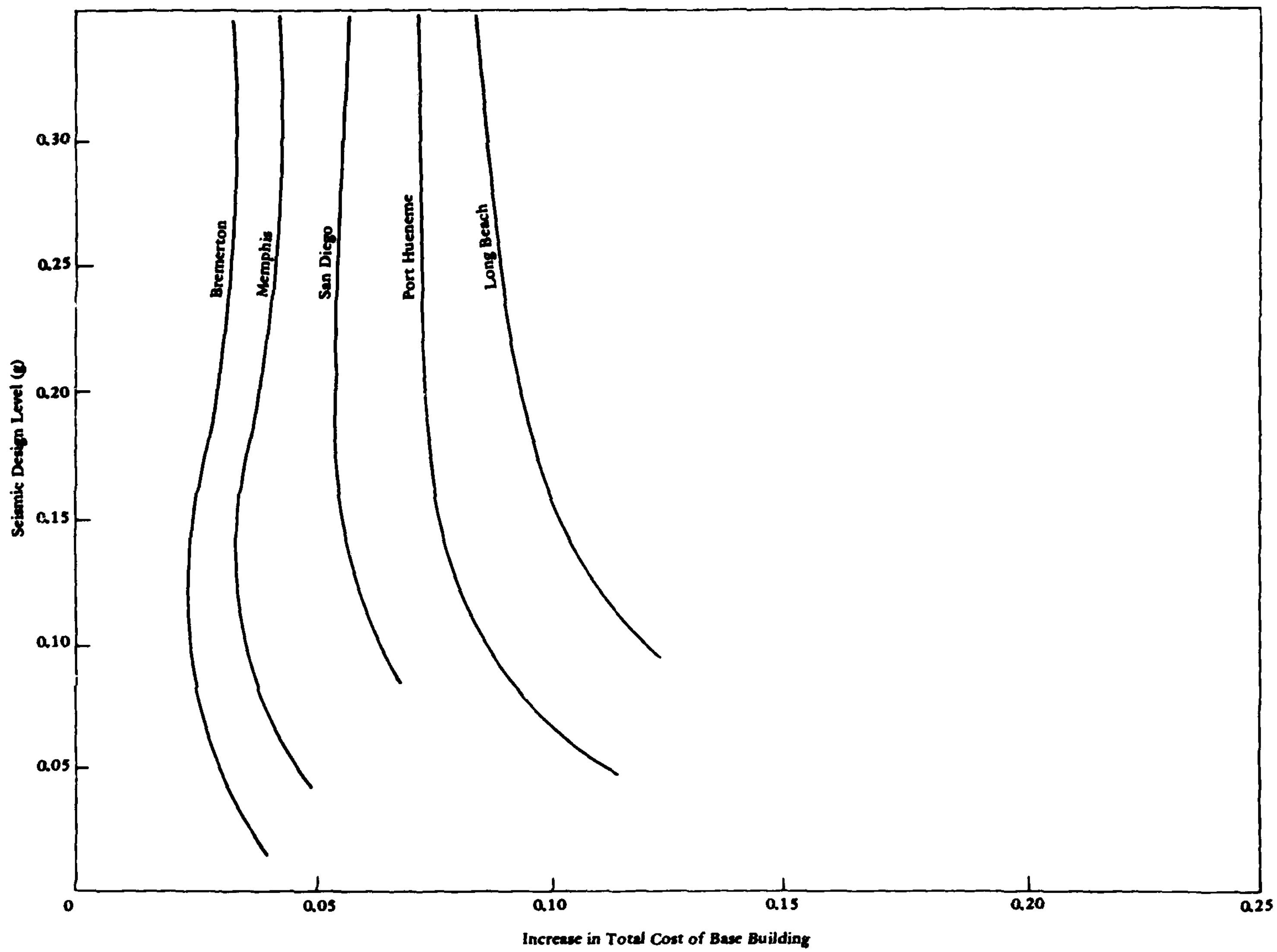


Figure 9. Increase in cost in various locations for shear wall structure.

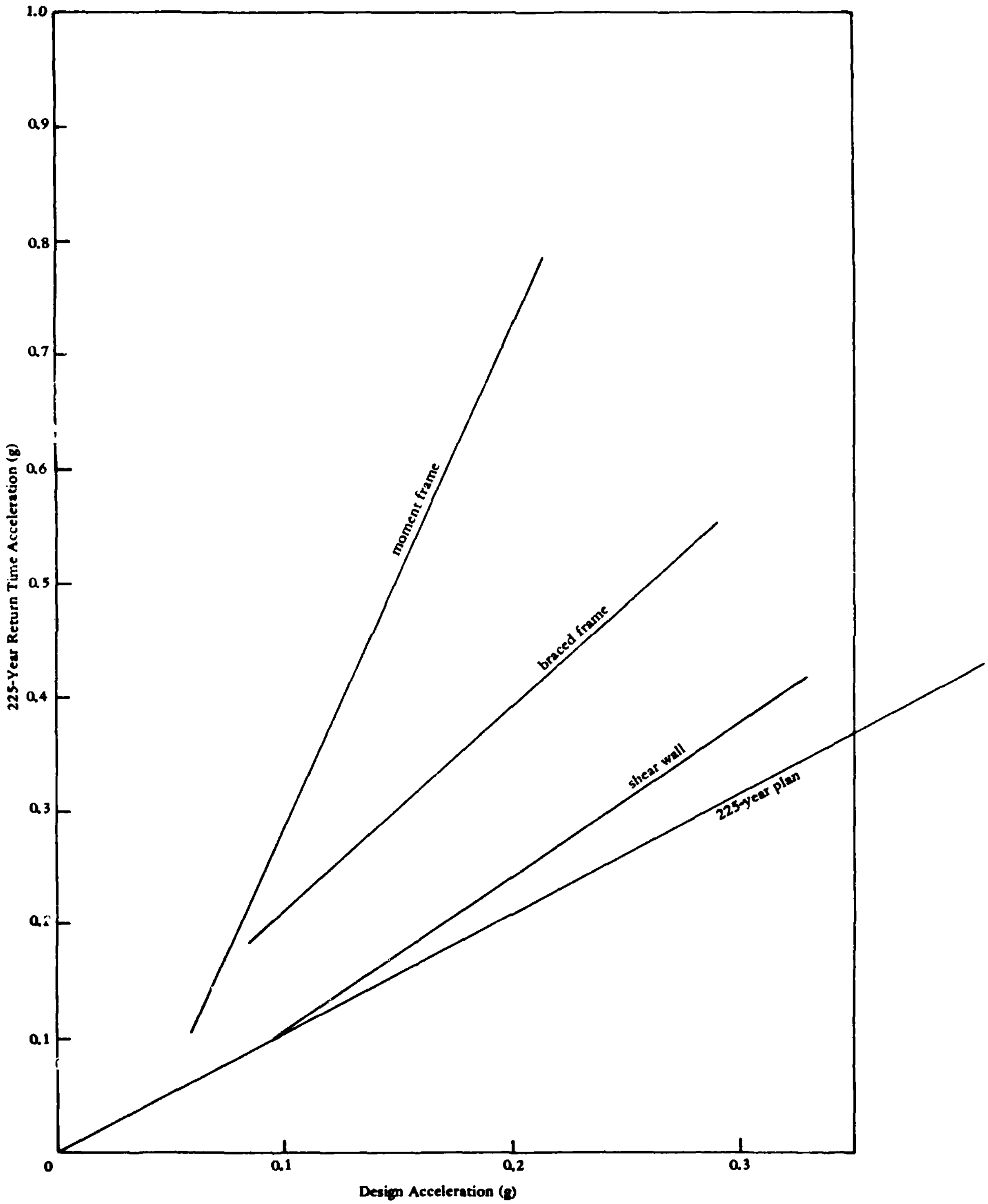


Figure 10. Least cost design acceleration, including present NAVFAC policy for important facilities.

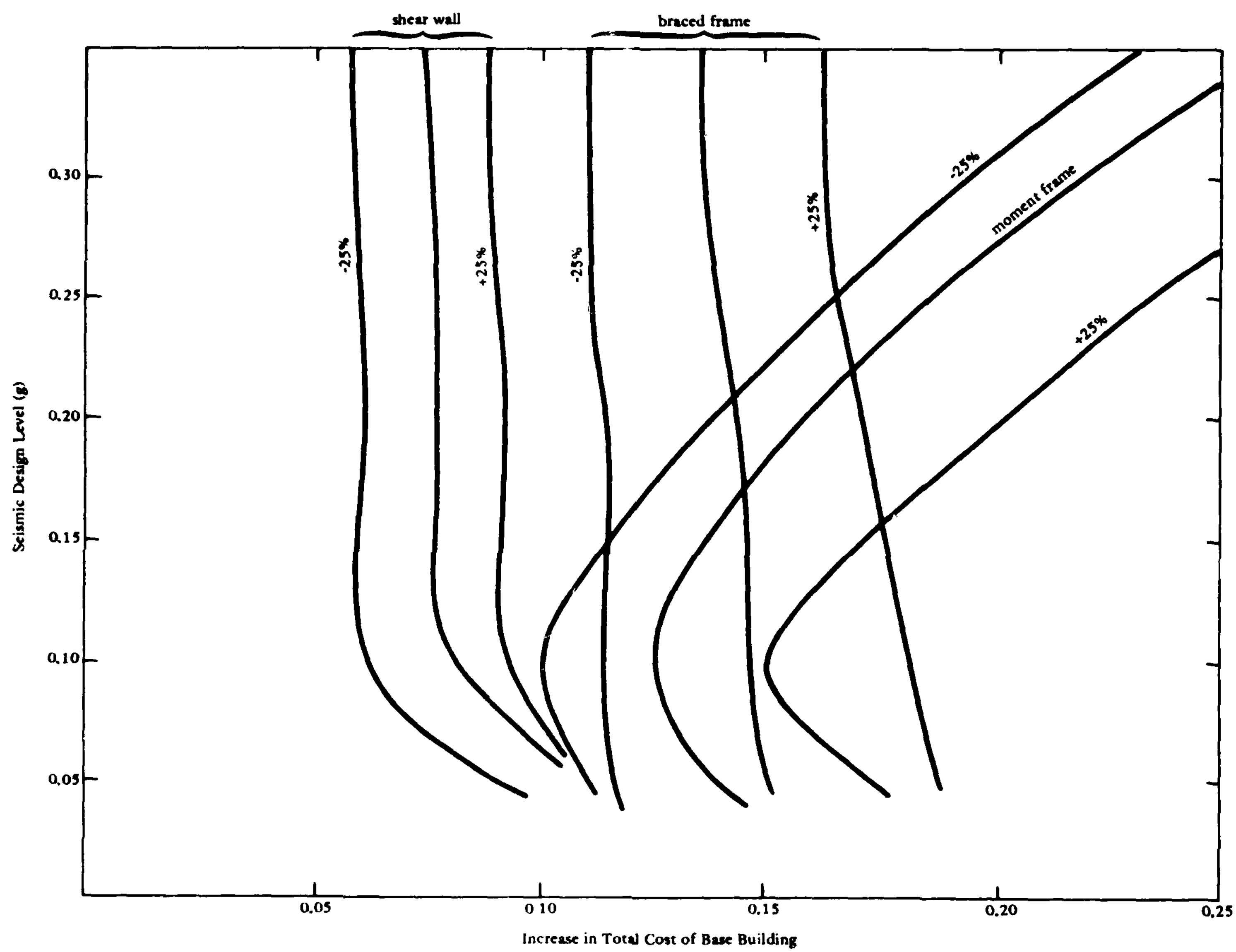


Figure 11. Effect of variation in damage estimates at Port Hueneme site.

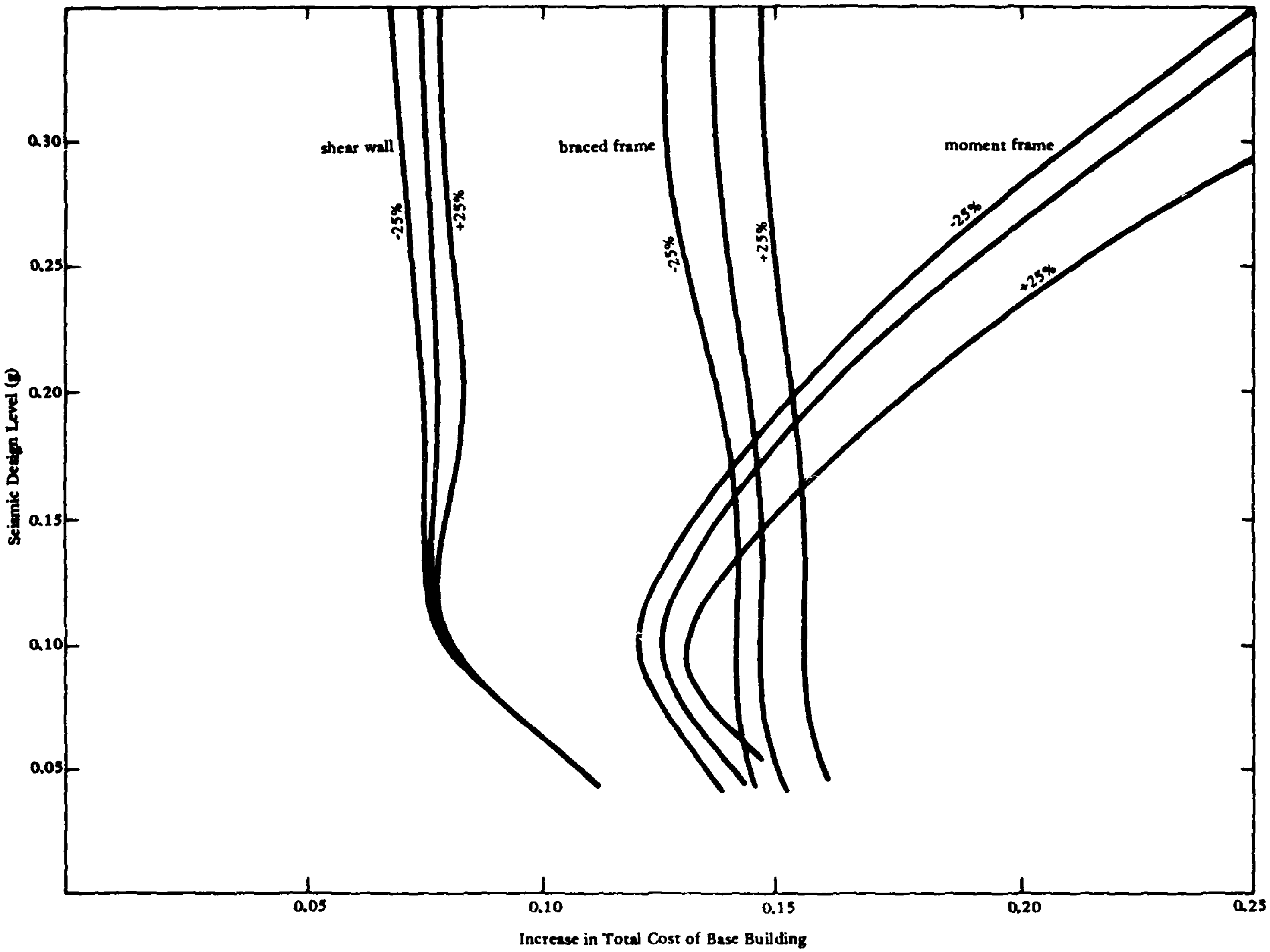


Figure 12. Effect of variation in cost estimates at Port Hueneme site.

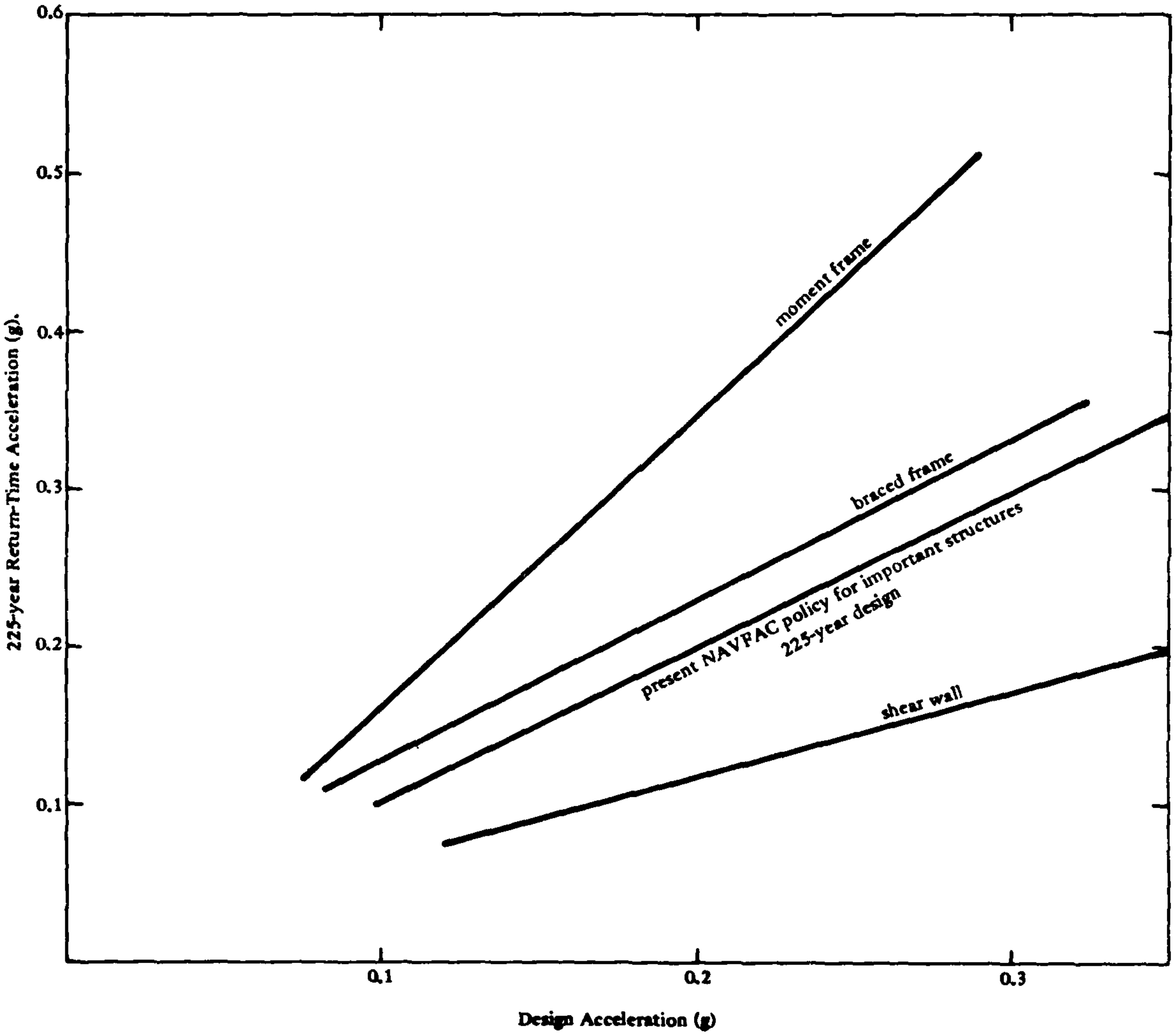


Figure 13. Design acceleration with damage penalty of 5.

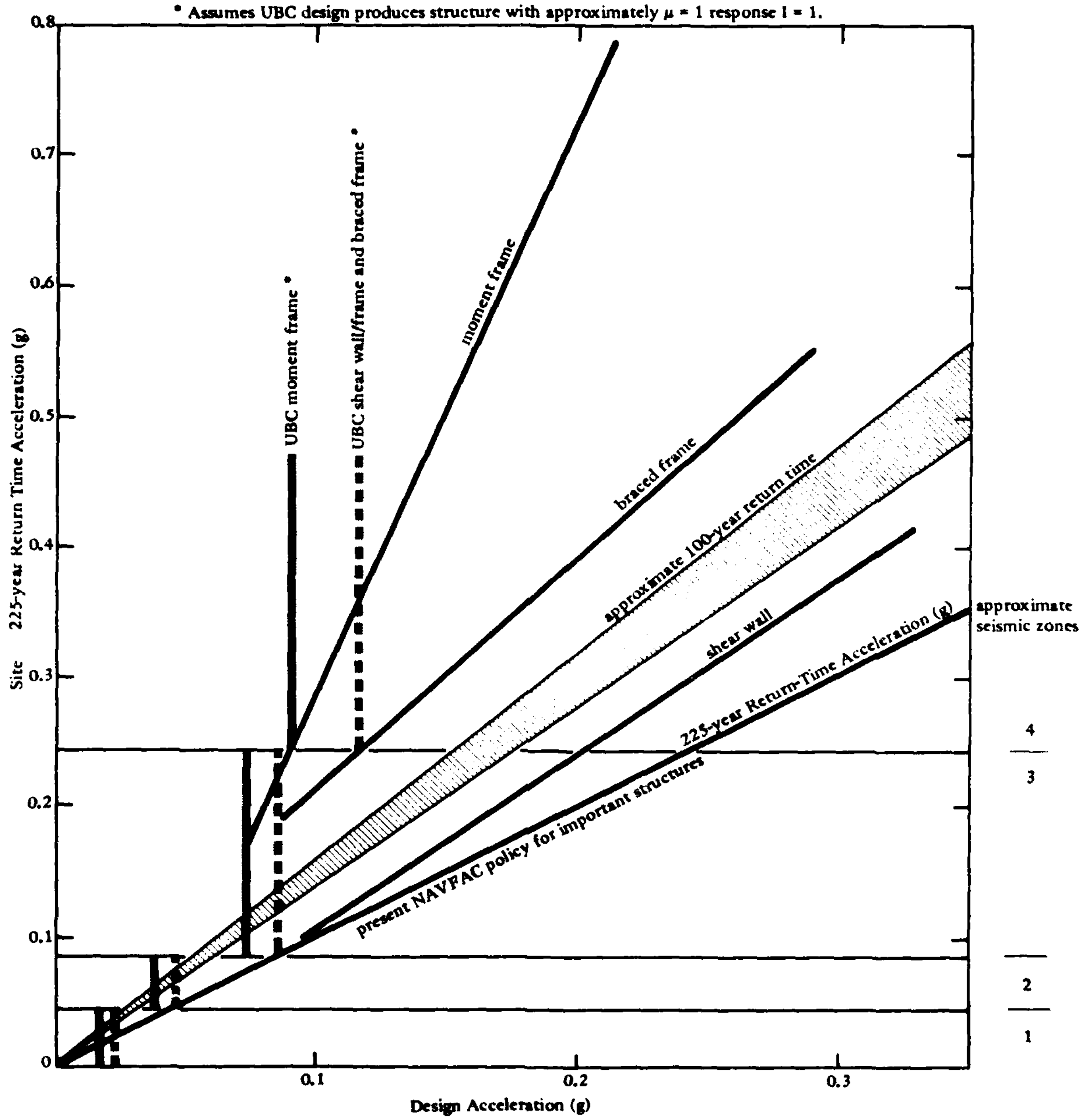


Figure 14. Least cost design acceleration for important structures (ductility = 1).

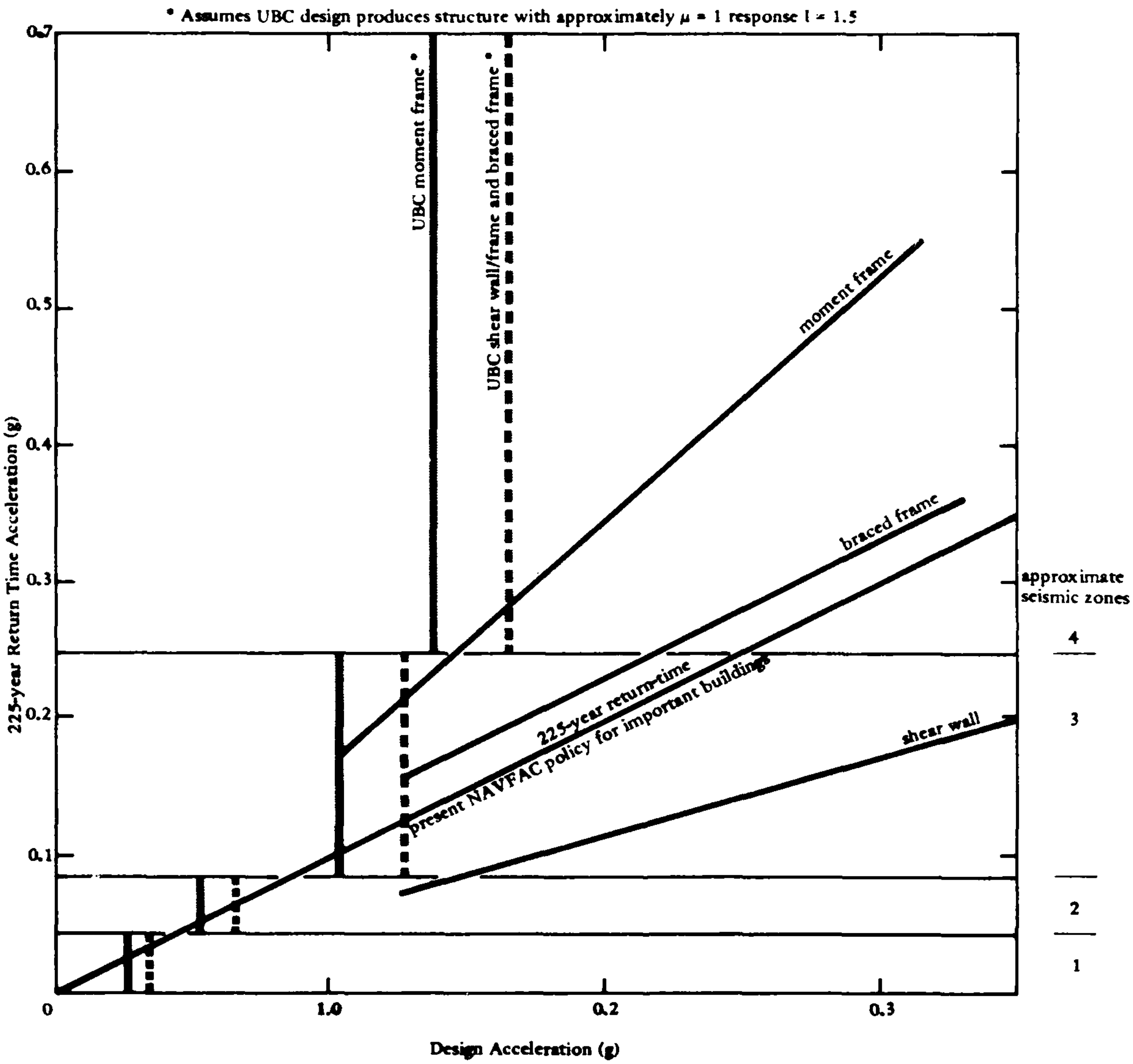


Figure 15. Least cost design acceleration for very important structures (ductility = 1).

Appendix A

PLAN AND ELEVATION OF THREE-STORY BUILDING

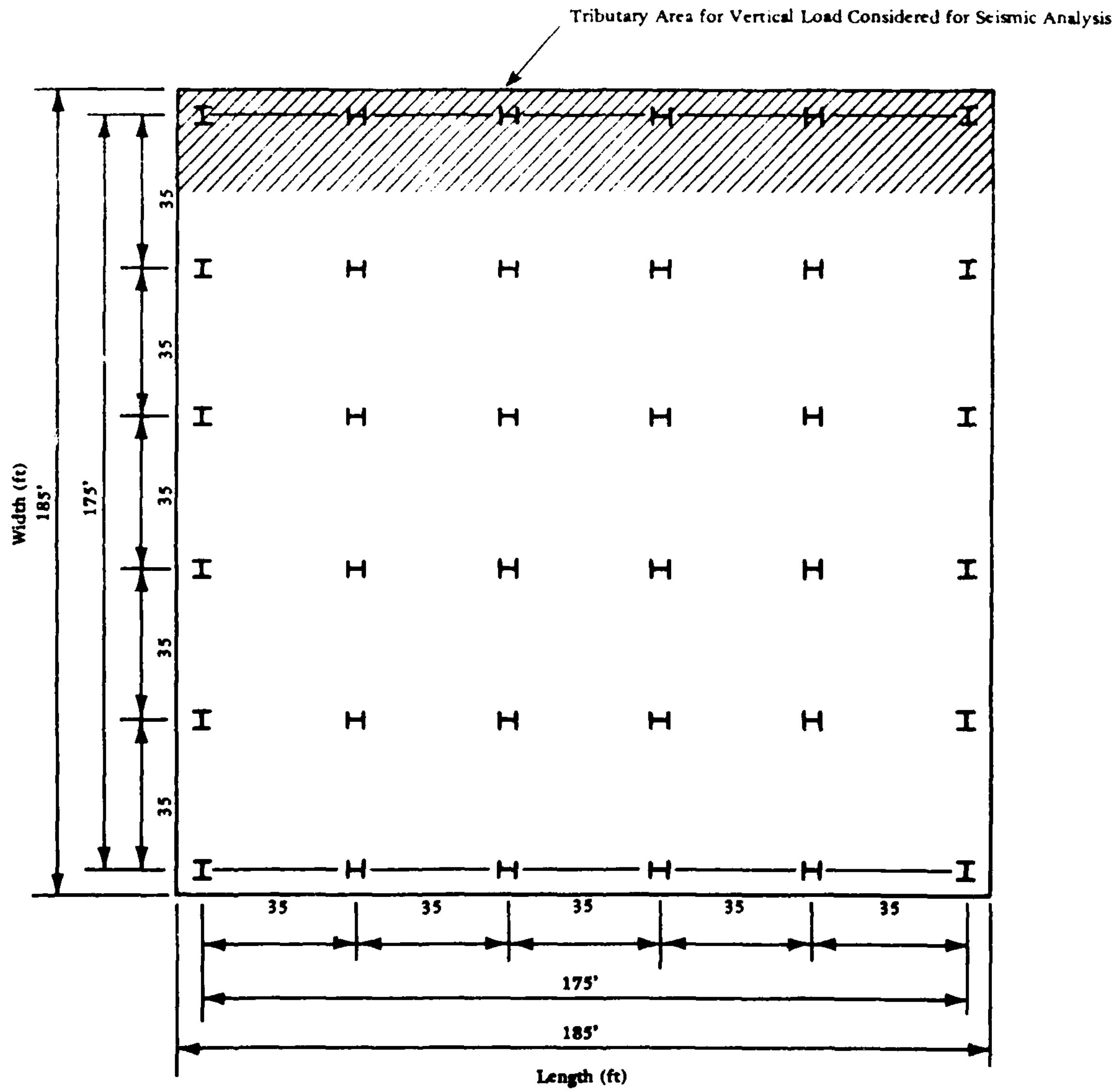


Figure A-1. Typical plan of three-story building.

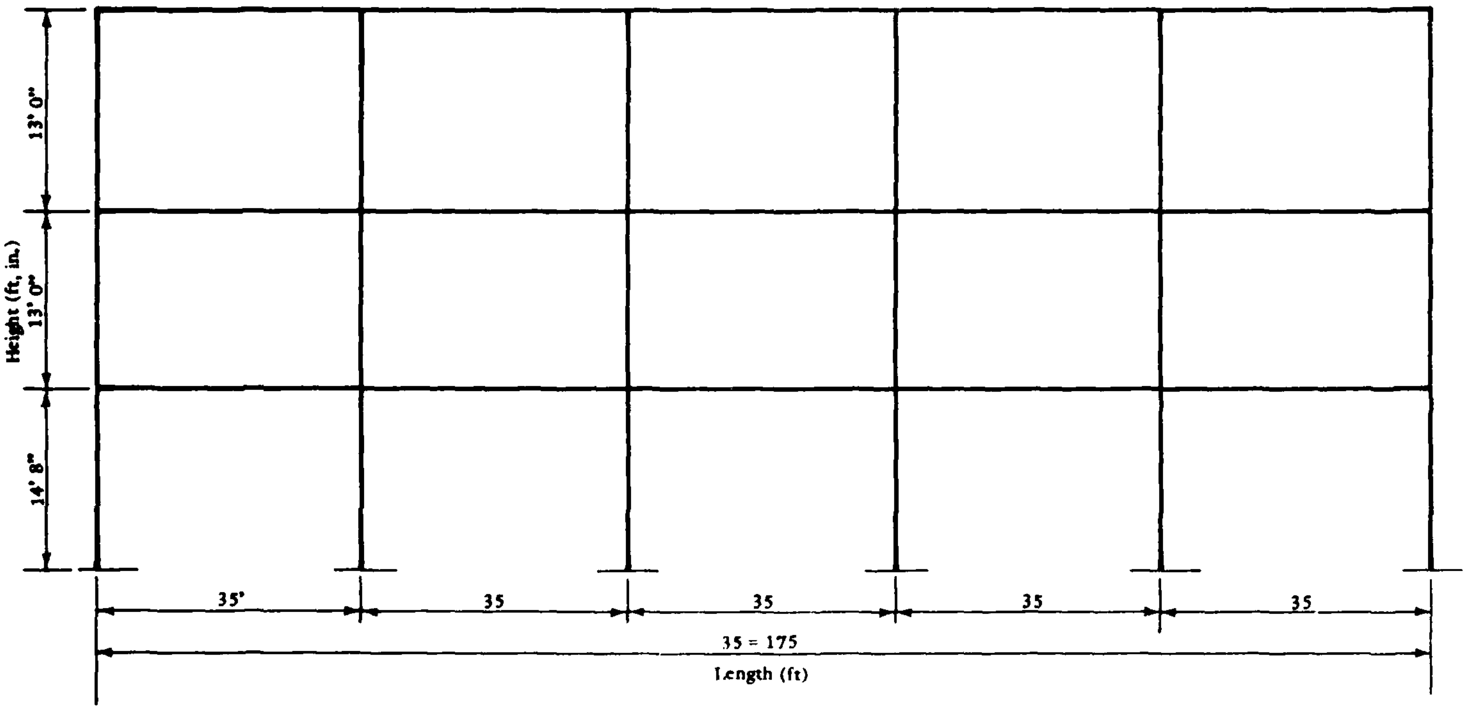


Figure A-2. Structural frame elevation.

Appendix B

STRUCTURE DESIGNS AND COSTS

by Walter Saunders
of
Martin & Saunders

INTRODUCTION

Three lateral force systems were selected for consideration in the study. The first of these is a ductile moment resistant steel rigid frame. The second is a braced frame in which the vertical bracing is in the form of an inverted vee. The third system is a shear wall system in which the walls are combined with a steel frame which carries the vertical load. Wall construction is either reinforced concrete or reinforced masonry depending upon the strength and framing requirements.

Elements of the various framing systems were proportioned using the strength design provision contained in the AISC Specifications (Ref B-1) and the ACI Building Code Requirements (Ref B-2). Design forces in the individual members were obtained by applying suitable load factors to the loads obtained from the dynamic response analysis. Load factors used in the design analysis are as follows: dead load plus live load, 1.7; dead load plus live load plus static lateral load, 1.3; and dead load plus live load plus seismic response spectrum, 1.0. Ultimate strength of the masonry was estimated as being twice the allowable value which is consistent with the work reported in Reference B-3.

DESIGN SPECTRUM

The design spectra used in this study are based on the elastic design spectrum developed by Newmark and Hall (Ref B-3). The six levels of base acceleration which include 10%g, 15%g, 20%g, 25%g, 30%g, and 35%g form the basis for six load conditions which will be applied to each of the three lateral force systems. The design spectra for these six values of base acceleration were constructed using the applicable amplification factors for 5% of structural damping. These spectra must be digitized so as to represent the spectrum in a linear manner in the dynamic response analysis. These data are used in the modal response analysis.

DESIGN ANALYSIS

Prior to beginning the preliminary design analysis, it is necessary to perform a detailed weight analysis of the original structure in order to determine the mass distribution in the structure and the distribution of gravity load to the peripheral frames. It is assumed that one-half of the total mass is taken by each of the peripheral frames which are parallel to the direction of the ground motion. In order to insure that relative story displacements were within acceptable bounds, it was necessary to establish drift criteria for the frames which were related to the seismic design spectra. It was therefore decided to use the criteria set forth in the ATC-3 Report (Ref B-4). This required defining the Effective Peak Velocity (EPV) as the maximum spectral velocity (obtained from the response spectrum) divided by 2.5. The velocity related acceleration coefficient (Av) was then obtained from the established relation between EPV and Av. With Av determined, the seismic coefficient could be calculated and used to determine the base shear. This is then increased by a suitable deflection coefficient to obtain the pseudo static forces to be used for evaluating drift. The maximum relative story displacement divided by story height according to this criteria is 0.015.

Preliminary design analyses of the moment frames and the braced frames were done using the computer program TABSSD (Three Dimensional Analysis of Building Systems with Strength Design). This program is an extension of the ETABS Program (Ref B-5) which includes strength design of steel members. Various subroutines have been added which incorporate the strength design provisions found in Part 2 of the AISC Specification. Also, tables of section properties have been added which allows the program to automatically select member sizes. To initiate the design process a preliminary design is required. In this case, the preliminary design was assumed to be similar to the design of the original reference building. The program then recursively performs a force analysis and member selection until there is no significant change in the member sizes. This is particularly significant in the case of a response spectrum analysis because the inertia loads are changing with each iteration.

Following the preliminary design analysis, the designed frames were reviewed and certain changes were made to reflect framing consideration, connections, and column splicing. In the moment frames it was considered necessary to incorporate a grade beam at the foundation line in order to realistically transmit the column base moments into the foundation. Also, it was decided to splice the columns just above the second floor level on all of the frames. For the braced frames, it was decided to use simple connections for the girders in the unbraced bays. Also from the preliminary analysis, it was determined that it would only be necessary to brace two of the five bays.

Based on the preliminary analysis of the braced frames, a preliminary design for the shear wall frames was determined. It was decided to place shear walls in only two bays so as to be compatible with the braced frame design and also the functional requirements for the structure. Preliminary analysis indicated that reinforced concrete walls would have to be used for the four highest base accelerations whereas reinforced masonry walls could be used for the lower two base accelerations.

Figures B-1 through B-19 show the moment frames, brace frames, and shear wall frames for the various base accelerations.

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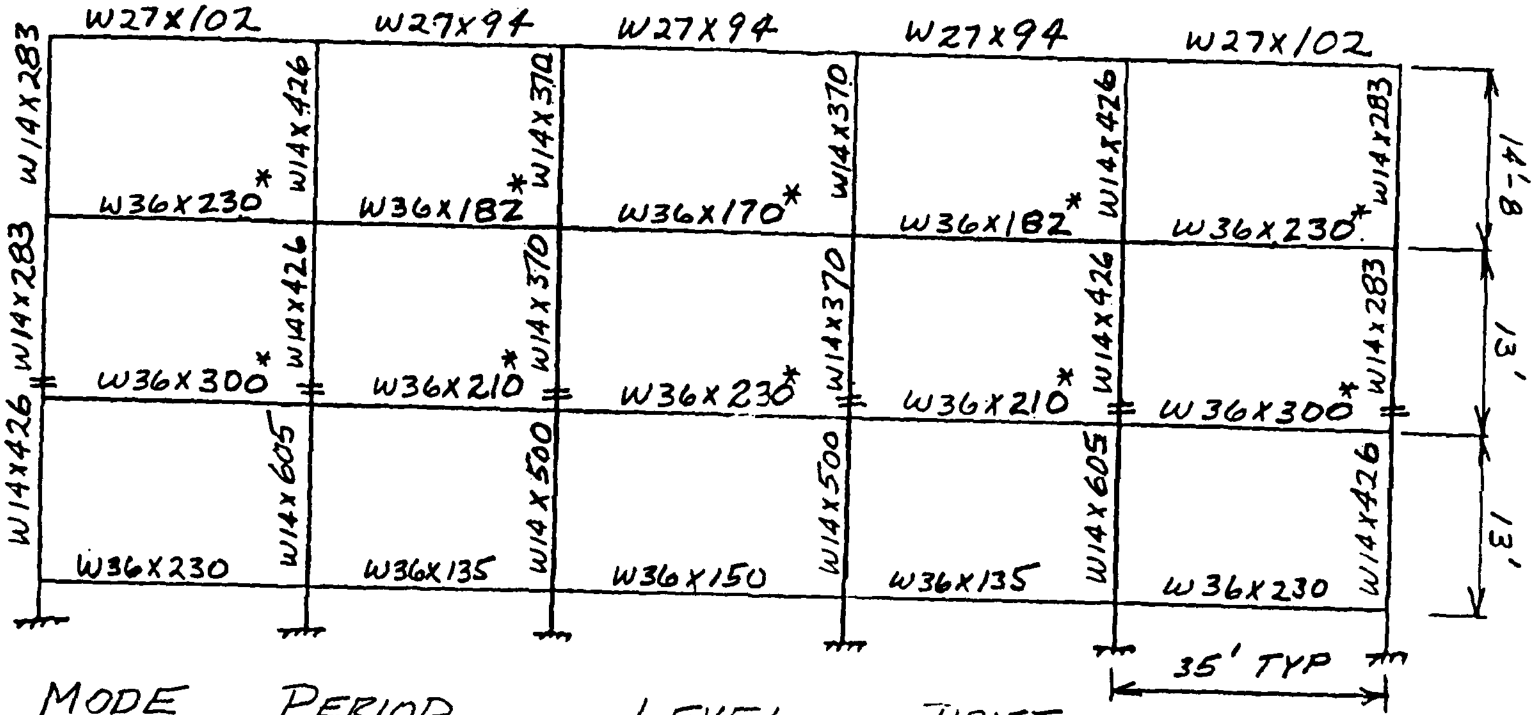
B-2. American Concrete Institute. ACI 318-77: Building code requirements for reinforced concrete. Detroit, Mich.

B-3. Department of Commerce, National Bureau of Standards. "Procedures and criteria for earthquake resistant design (Part II)," in Building Practices for Disaster Mitigation, BSS 46. Washington, DC, Feb 1973.

B-4. Applied Technology Council. NBS Special Publication 510: Tentative provisions for the development of seismic regulations for buildings. Jun 1978. (ATC 3-06)

B-5. University of California, Berkeley, Earthquake Engineering Research Center. Report No. EERC 75-13: Three dimensional analysis of building system (extended version), by E. L. Wilson et al. Berkeley, Calif., Apr 1975.

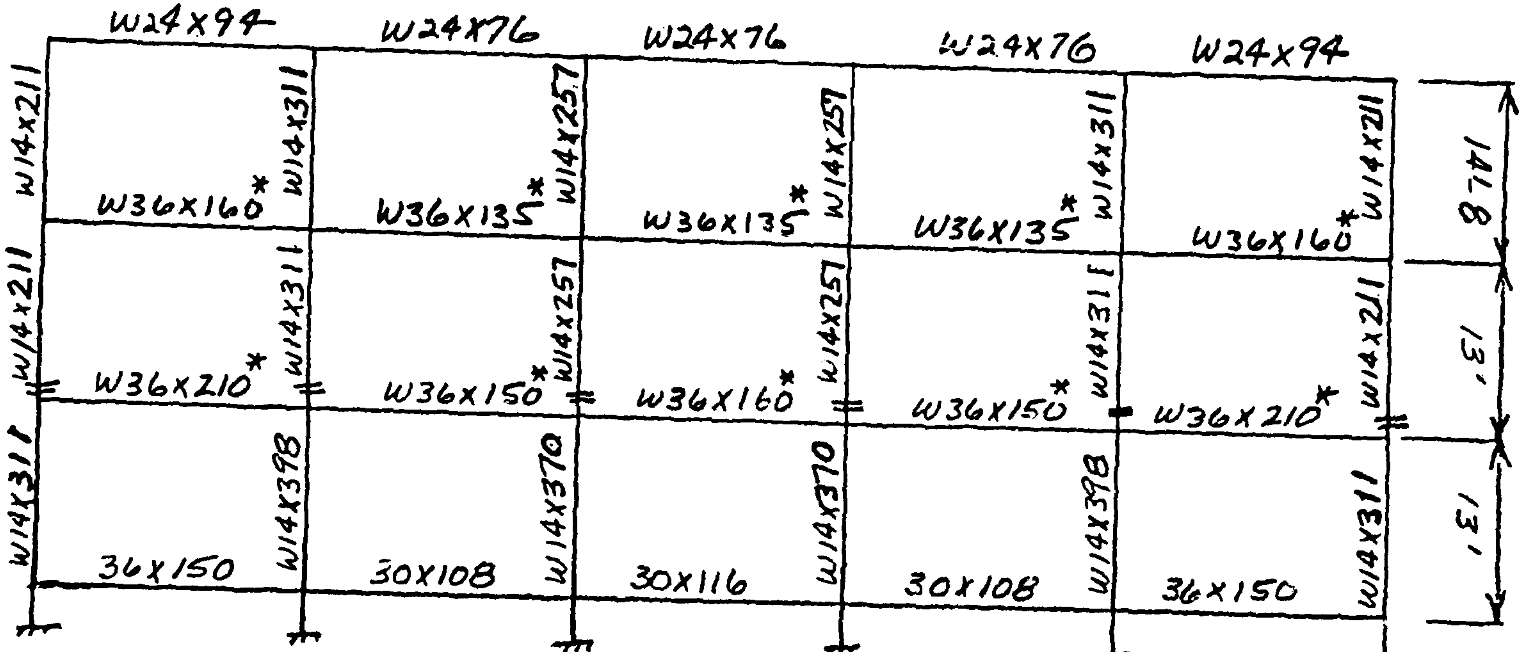


MODE	PERIOD
1	.607
2	.249
3	.139

LEVEL	DRIFT
3	.0073
2	.0076
1	.0064

* A572
GRADE 42

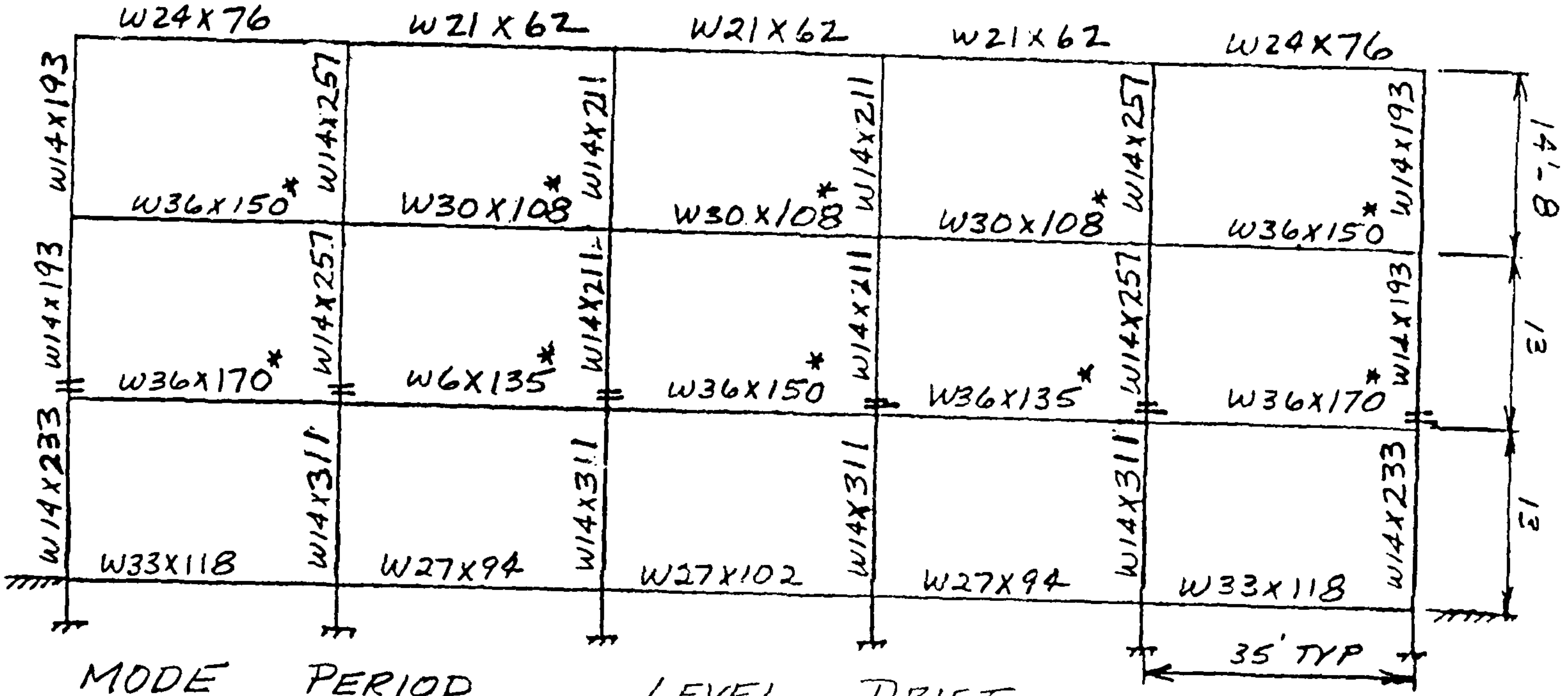
Figure B-1. Moment frame, 0.35g base acceleration.



MODE	PERIOD	LEVEL	DRIFT
1	.692	3	.0092
2	.289	2	.0096
3	.156	1	.0080

*A572 G42

Figure B-2. Moment frame, 0.30g base acceleration.

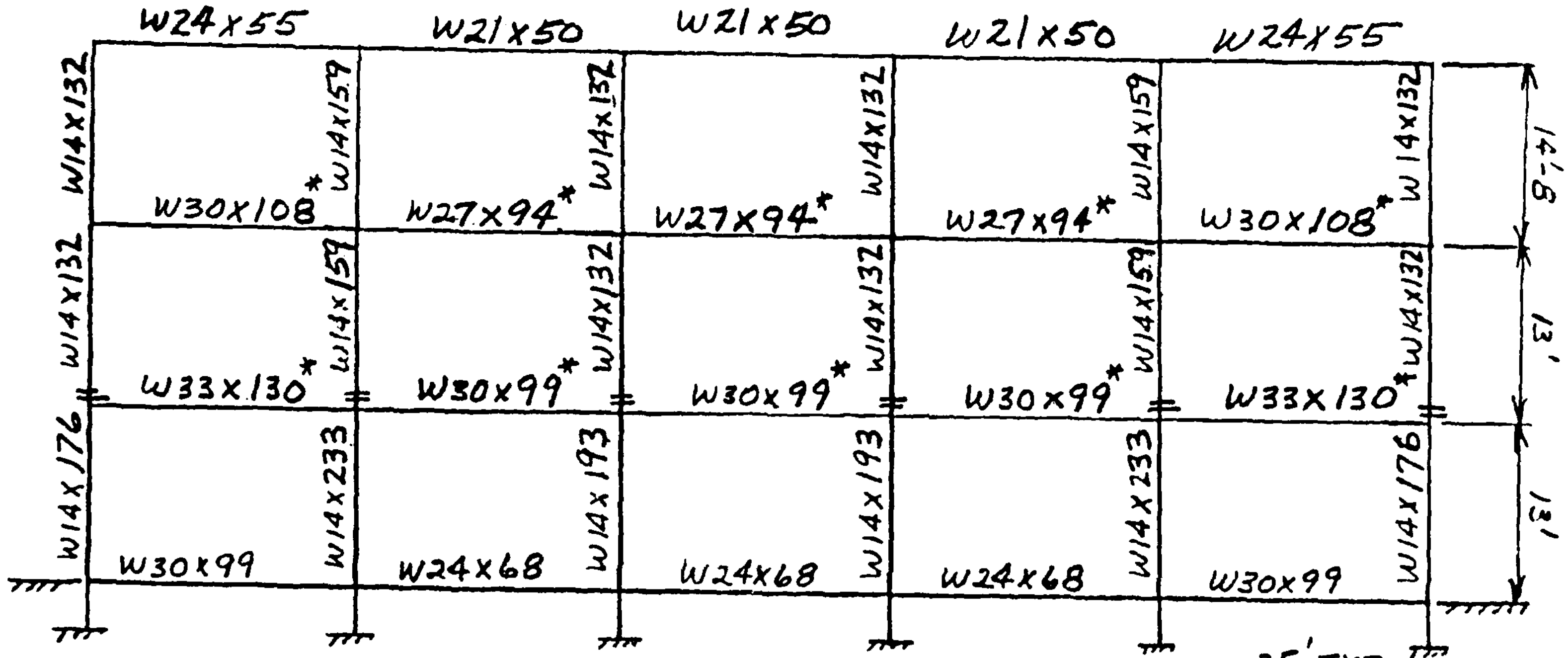


MODE	PERIOD
1	.776
2	.326
3	.179

LEVEL	DRIFT
3	.0099
2	.0105
1	.0080

* A572 GR 42

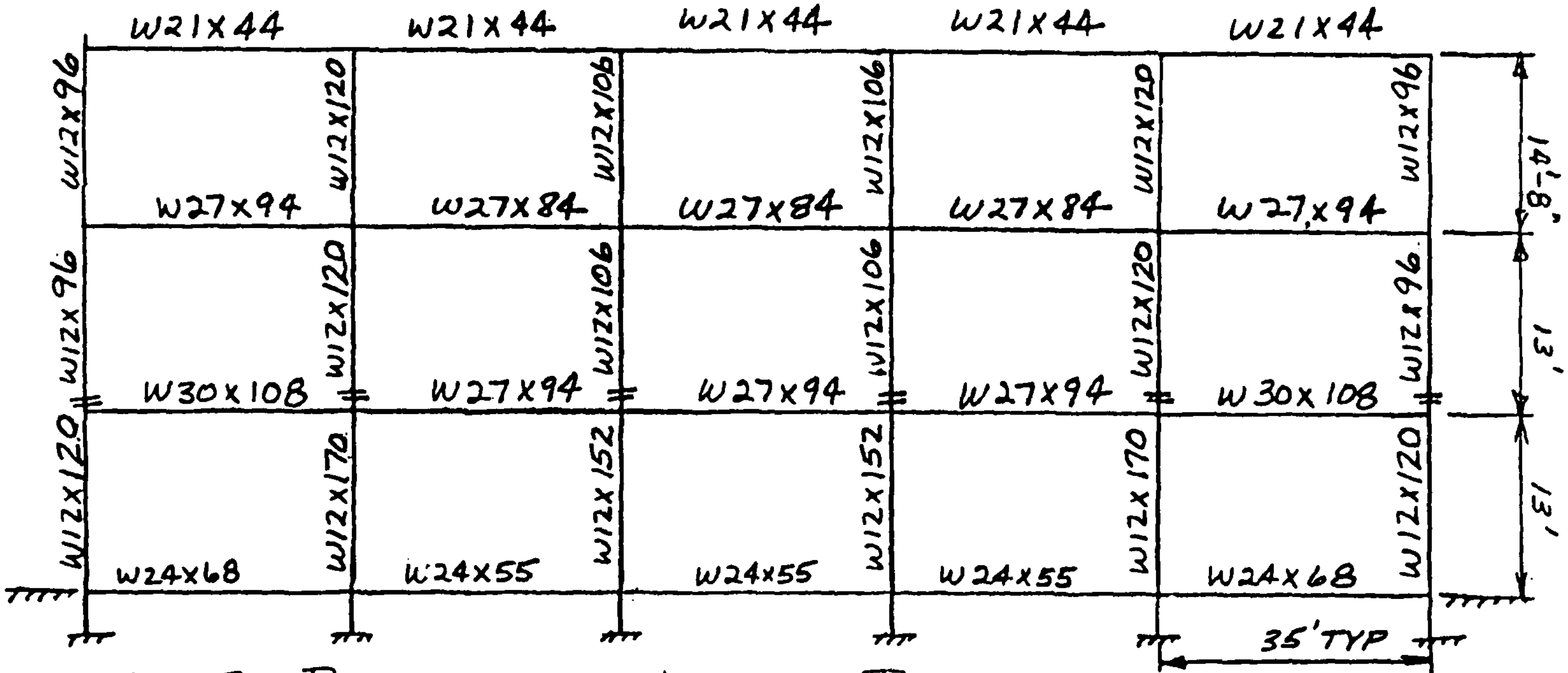
Figure B-3. Moment frame, 0.25g base acceleration.



MODE	PERIOD	LEVEL	DRIFT
1	1.039	3	.0136
2	.434	2	.0146
3	.238	1	.0121

A572 GR 42

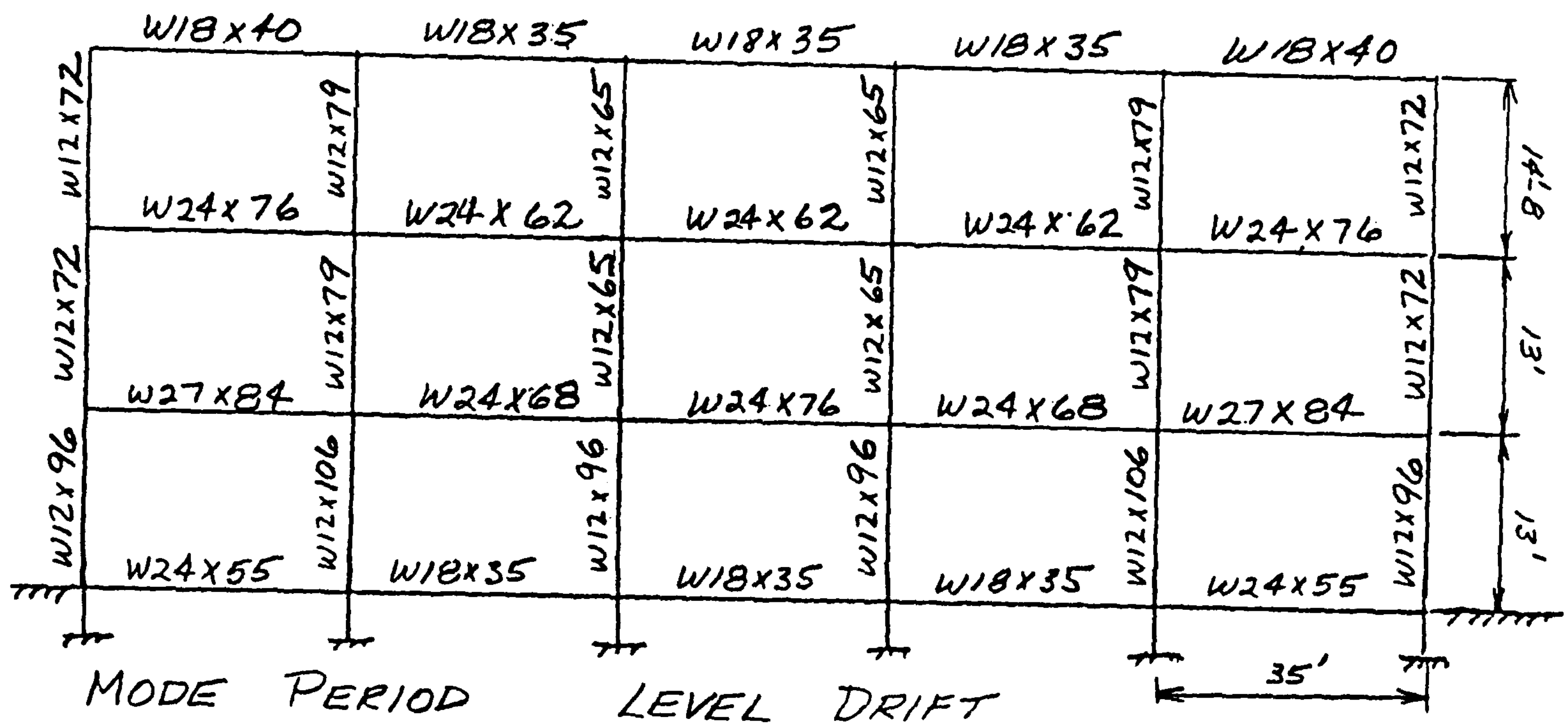
Figure B-4. Moment frame, 0.20g base acceleration.



MODE	PERIOD
1	1.220
2	.502
3	.286

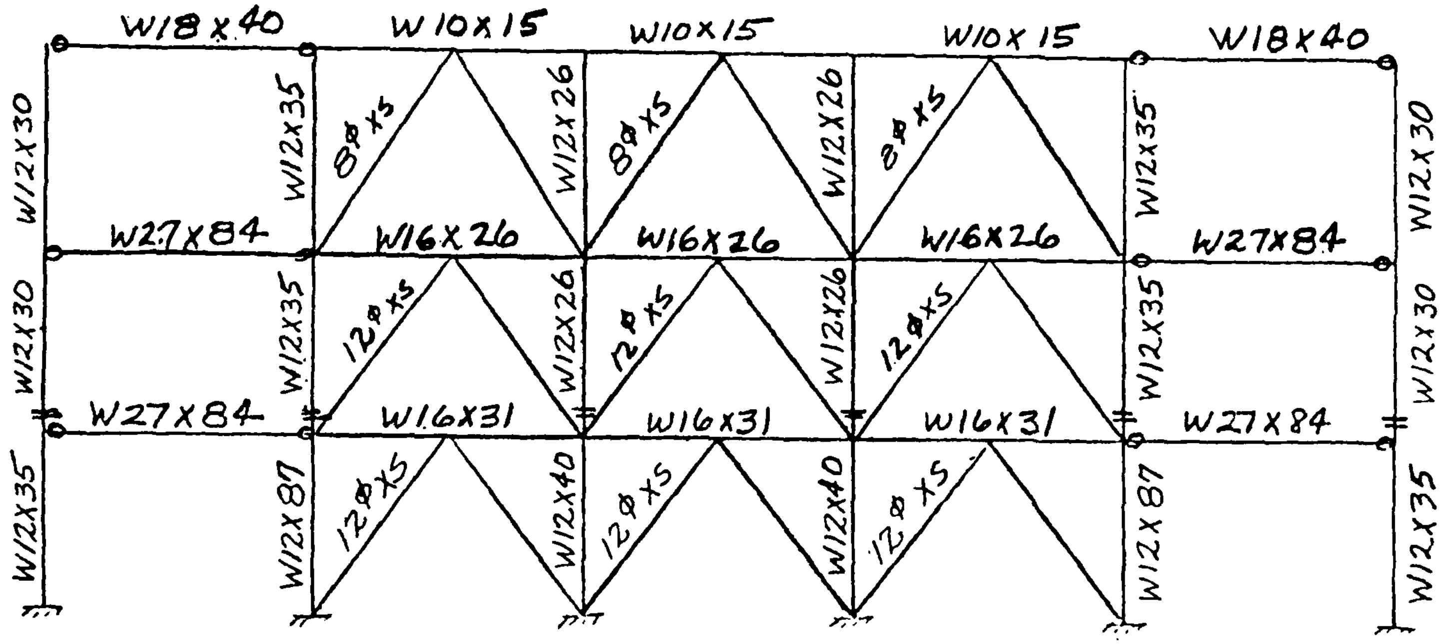
LEVEL	DRIFT
3	.0128
2	.0149
1	.0131

Figure B-5. Moment frame, 0.15g base acceleration.



MODE	PERIOD		LEVEL	DRIFT
1	1.519		3	.0121
2	.593		2	.0169
3	.359		1	.0127

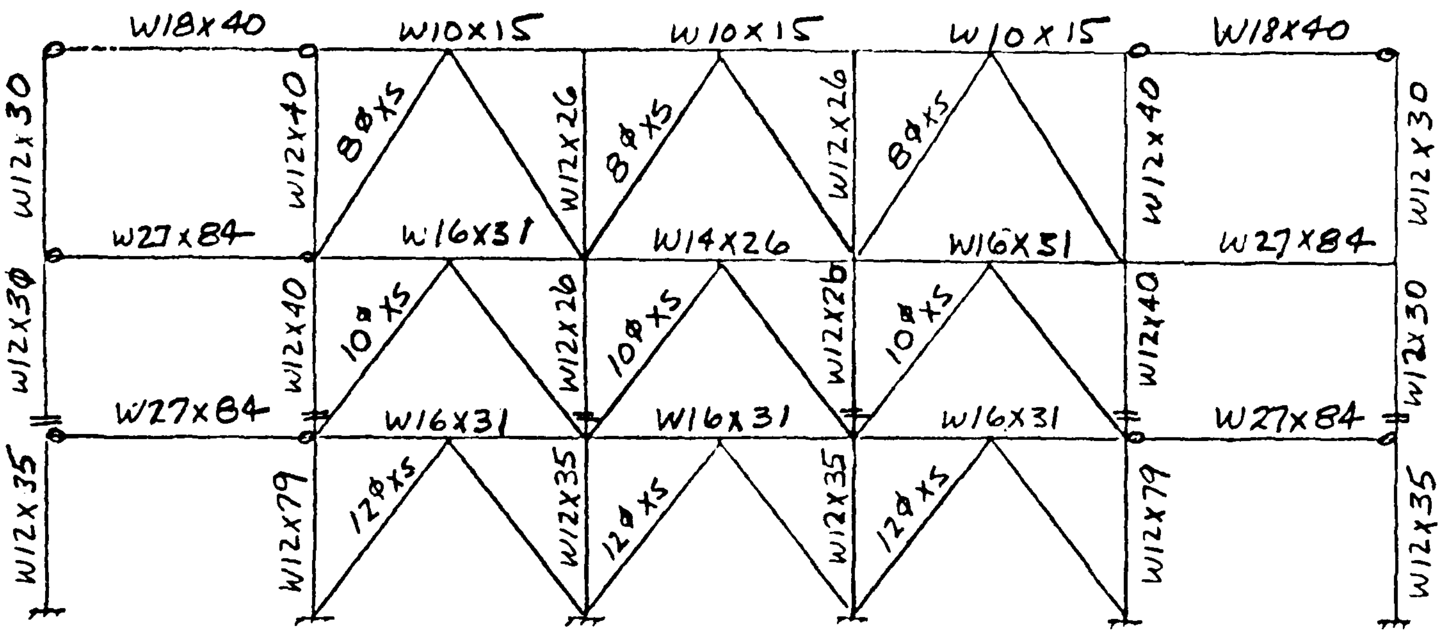
Figure B-6. Moment frame, 0.10g base acceleration.



MODE	PERIOD	LEVEL	DRIFT
1	.284	3	.00341
2	.121	2	.00486
3	.085	1	.00564

BRACING: $F_y = 46$ KSI

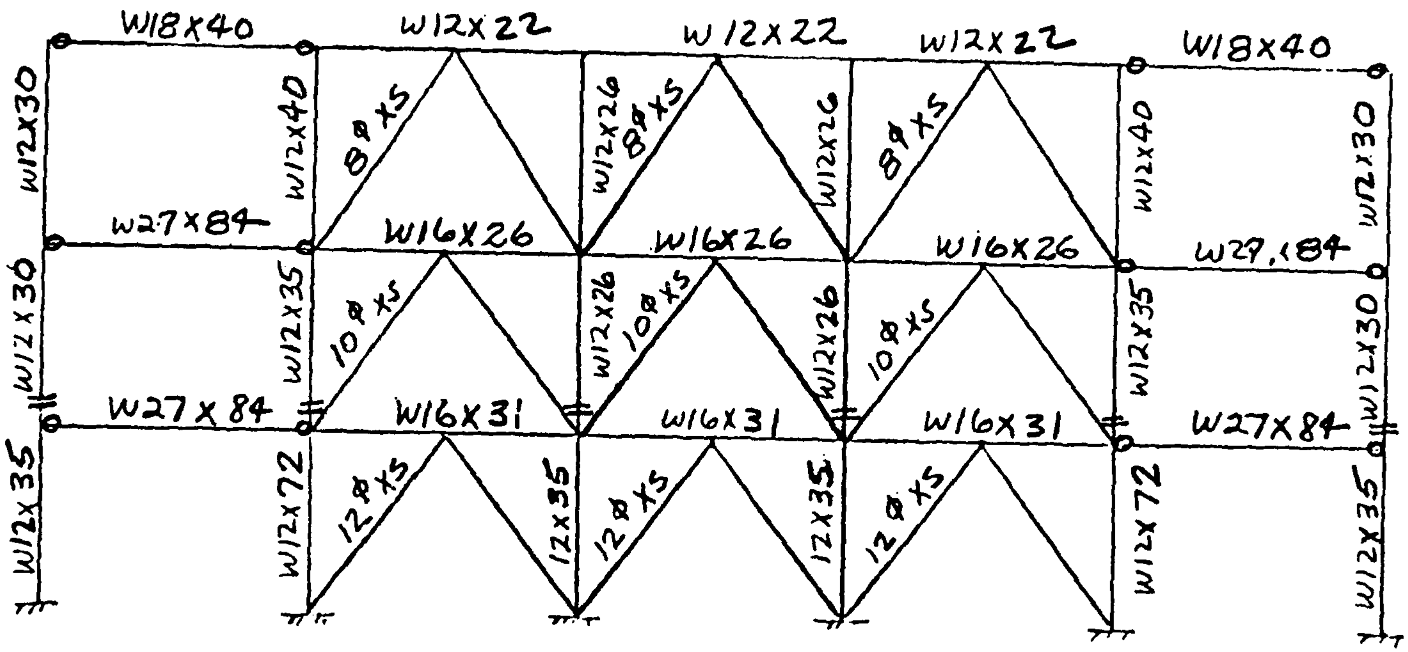
Figure B-7. Braced frame, 0.35g base acceleration.



MODE	PERIOD	LEVEL	DRIFT
1	.298	3	.00302
2	.124	2	.00422
3	.090	1	.00448

BRACING: $F_y = 46$ KSI

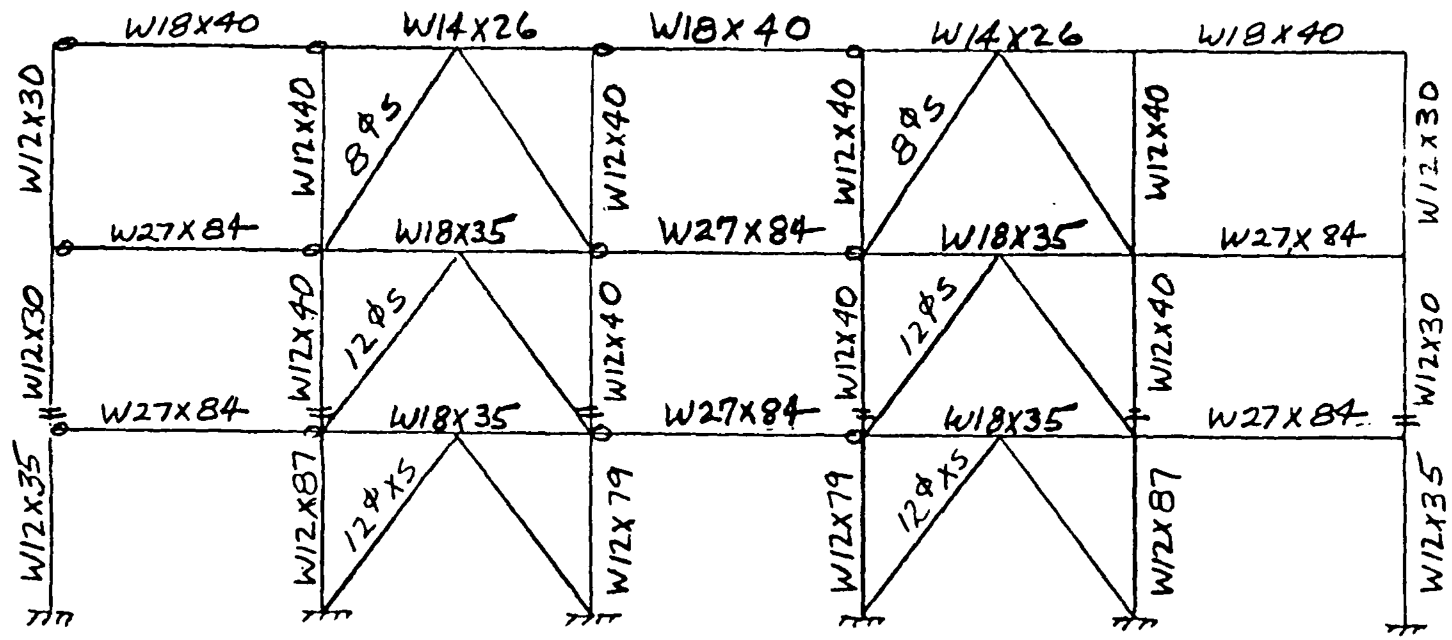
Figure B-8. Braced frame, 0.30g base acceleration.



MODE	PERIOD	LEVEL	DRIFT
1	.296	3	.0025
2	.123	2	.0041
3	.090	1	.0042

BRACING: $F_y = 46$ ksi

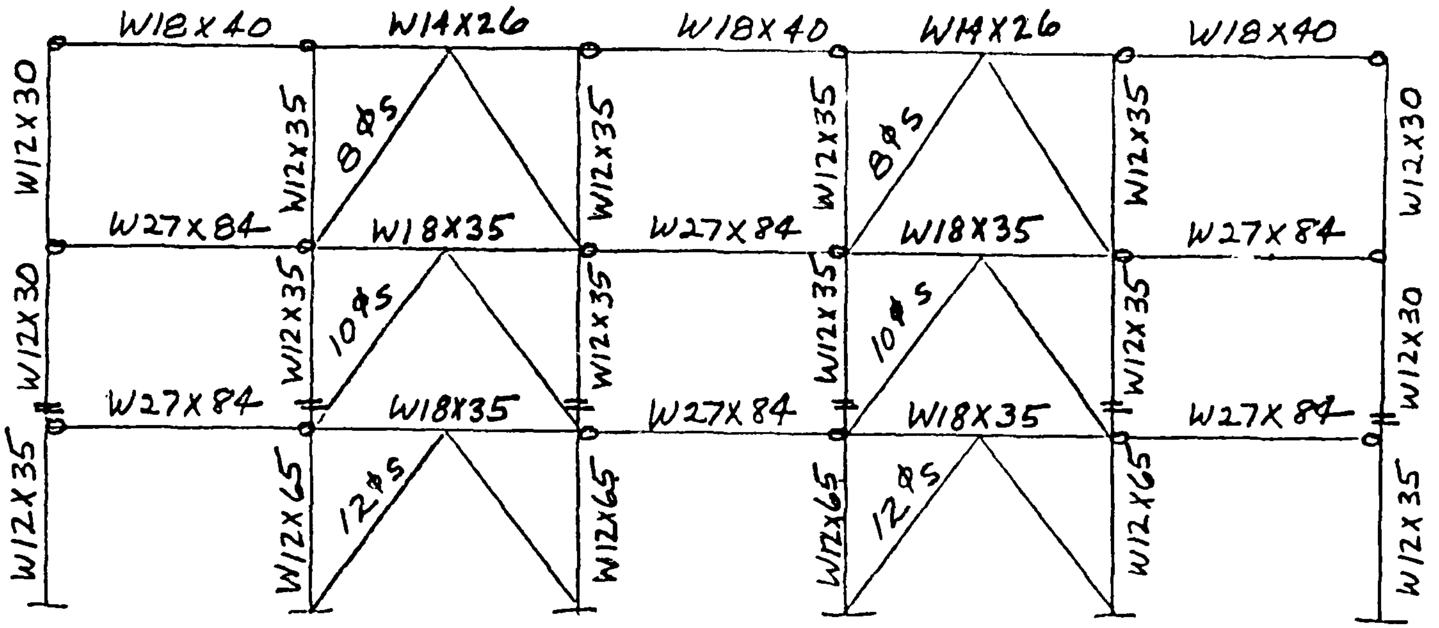
Figure B-9. Braced frame, 0.25g base acceleration.



MODE	PERIOD	LEVEL	DRIFT
1	.377	3	.0048
2	.172	2	.0054
3	.118	1	.0049

BRACING: $F_y = 46$ KSI

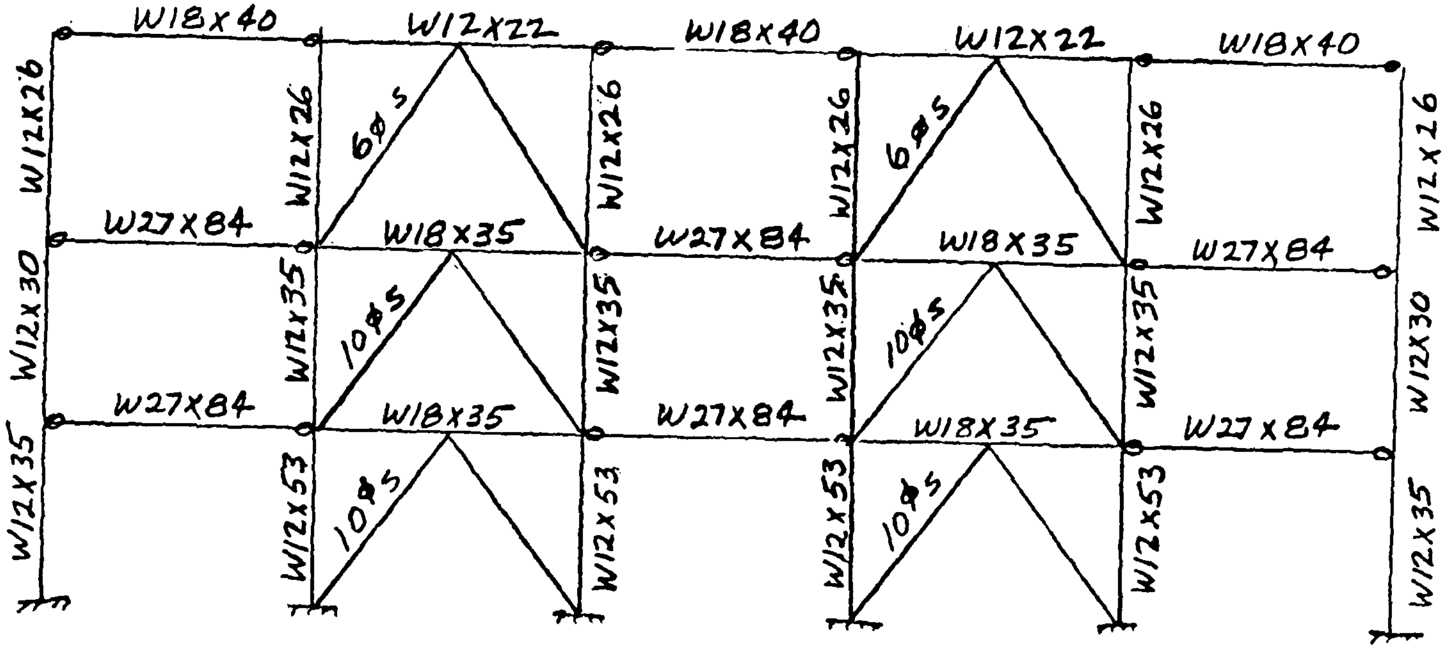
Figure B-10. Braced frame, 0.20g base acceleration.



MODE	PERIOD	LEVEL	DRIFT
1	.410	3	.0040
2	.184	2	.0050
3	.126	1	.0043

BRACING: $F_y = 46$ KSI

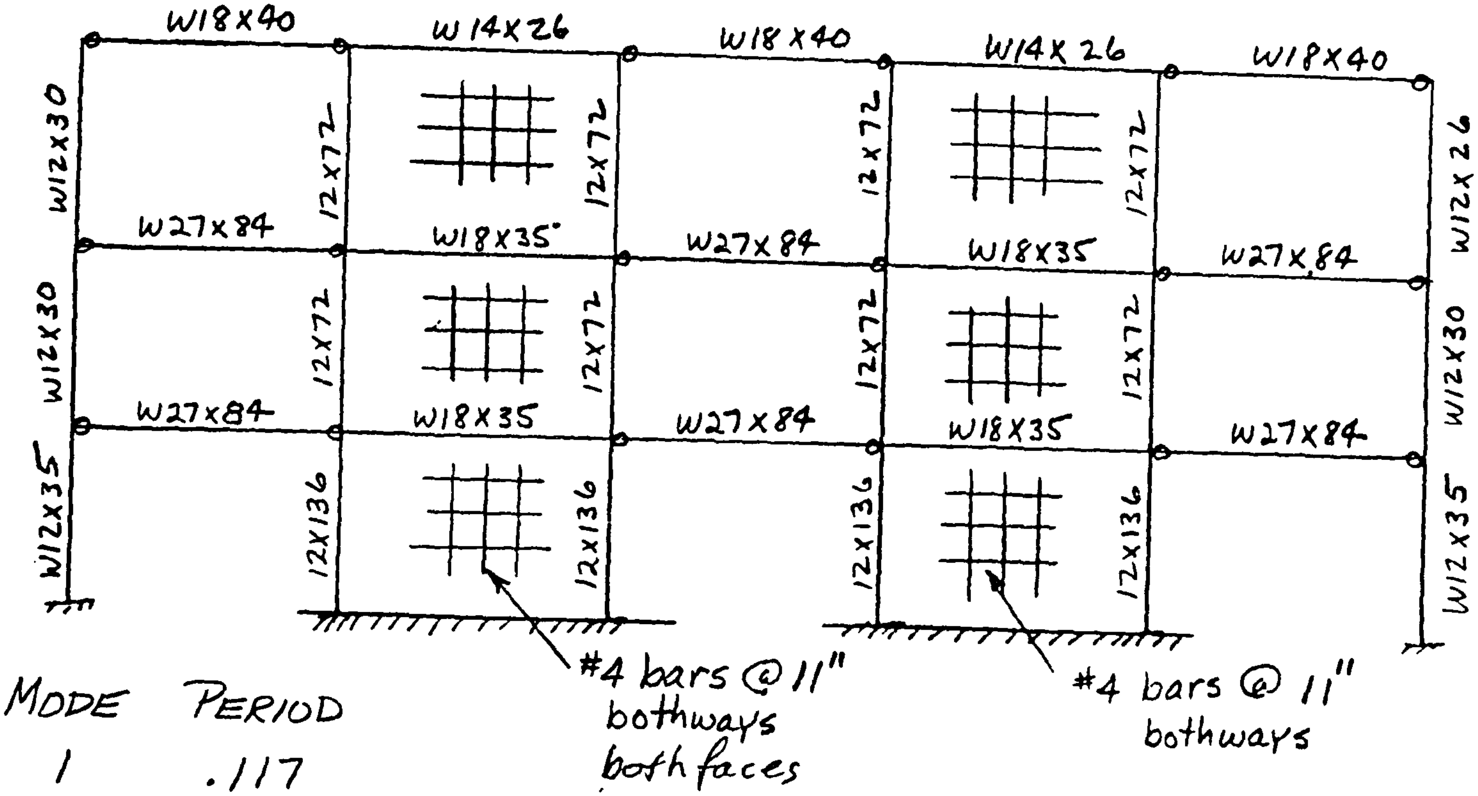
Figure B-11. Braced frame, 0.15g base acceleration.



MODE	PERIOD	LEVEL	DRIFT
1	.447	3	.0026
2	.191	2	.0035
3	.131	1	.0039

BRACING: $F_y = 46$ ksi

Figure B-12. Braced frame, 0.10g base acceleration.



MODE	PERIOD
1	.117
2	.041
3	.025

Figure B-13. Shear wall frame, 0.35g base acceleration, 14-inch concrete wall.

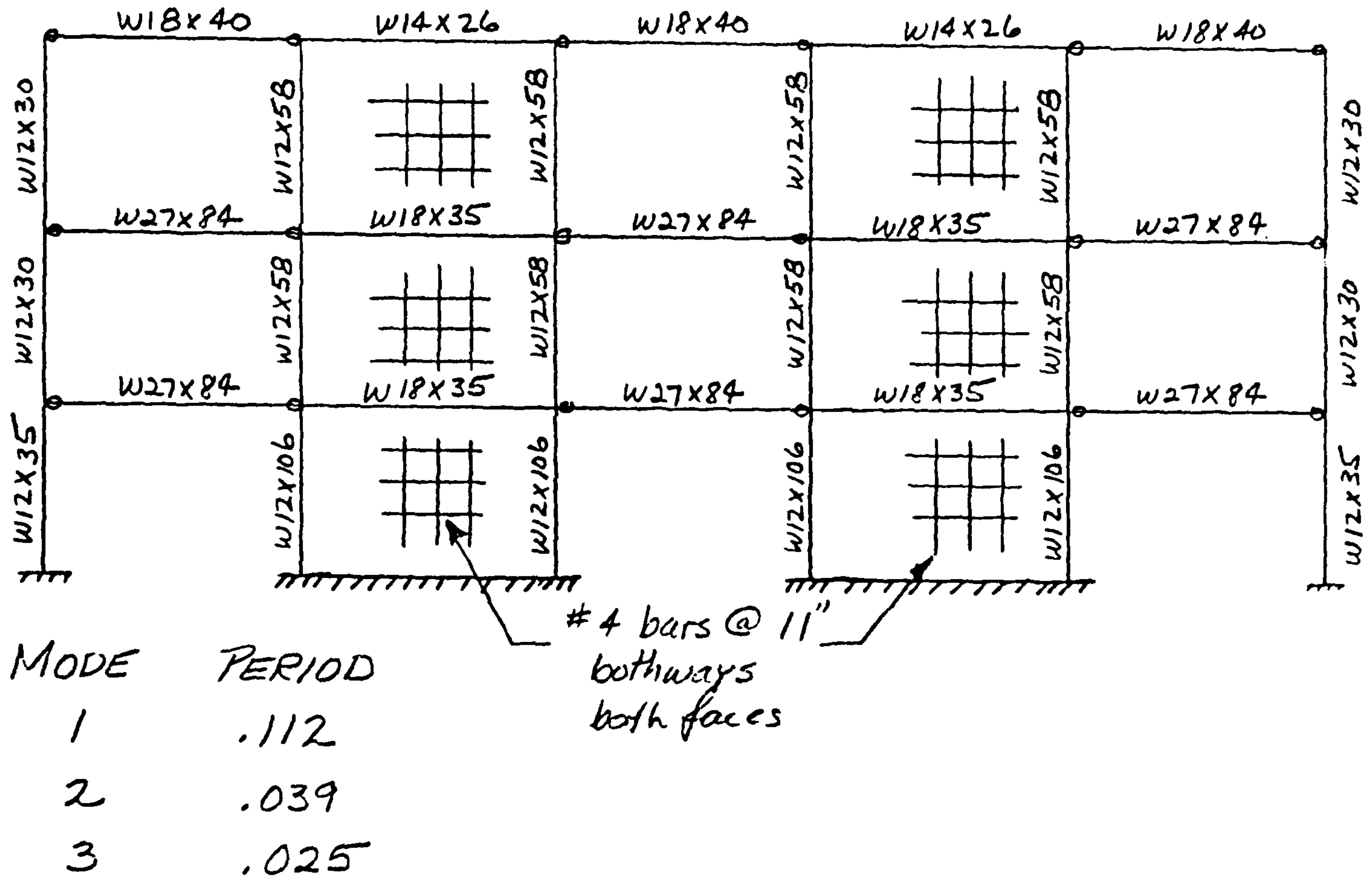
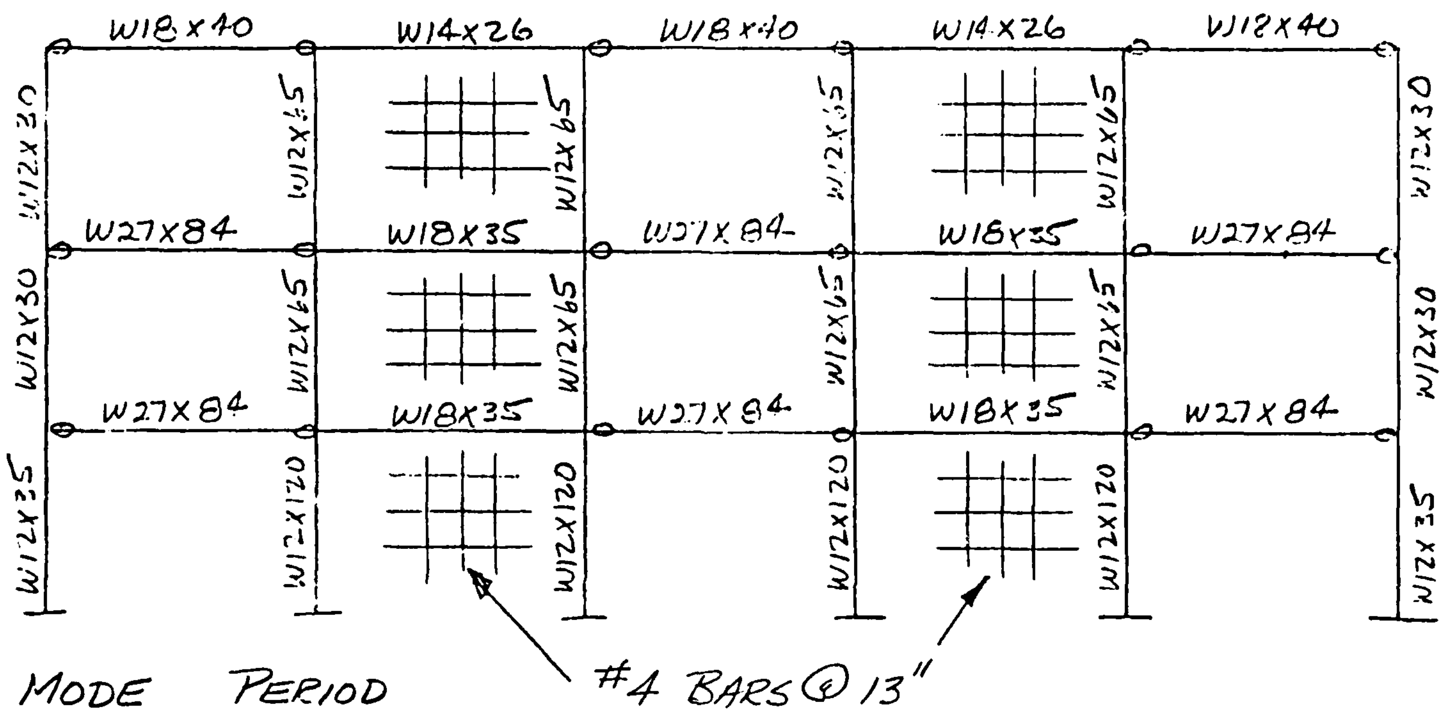
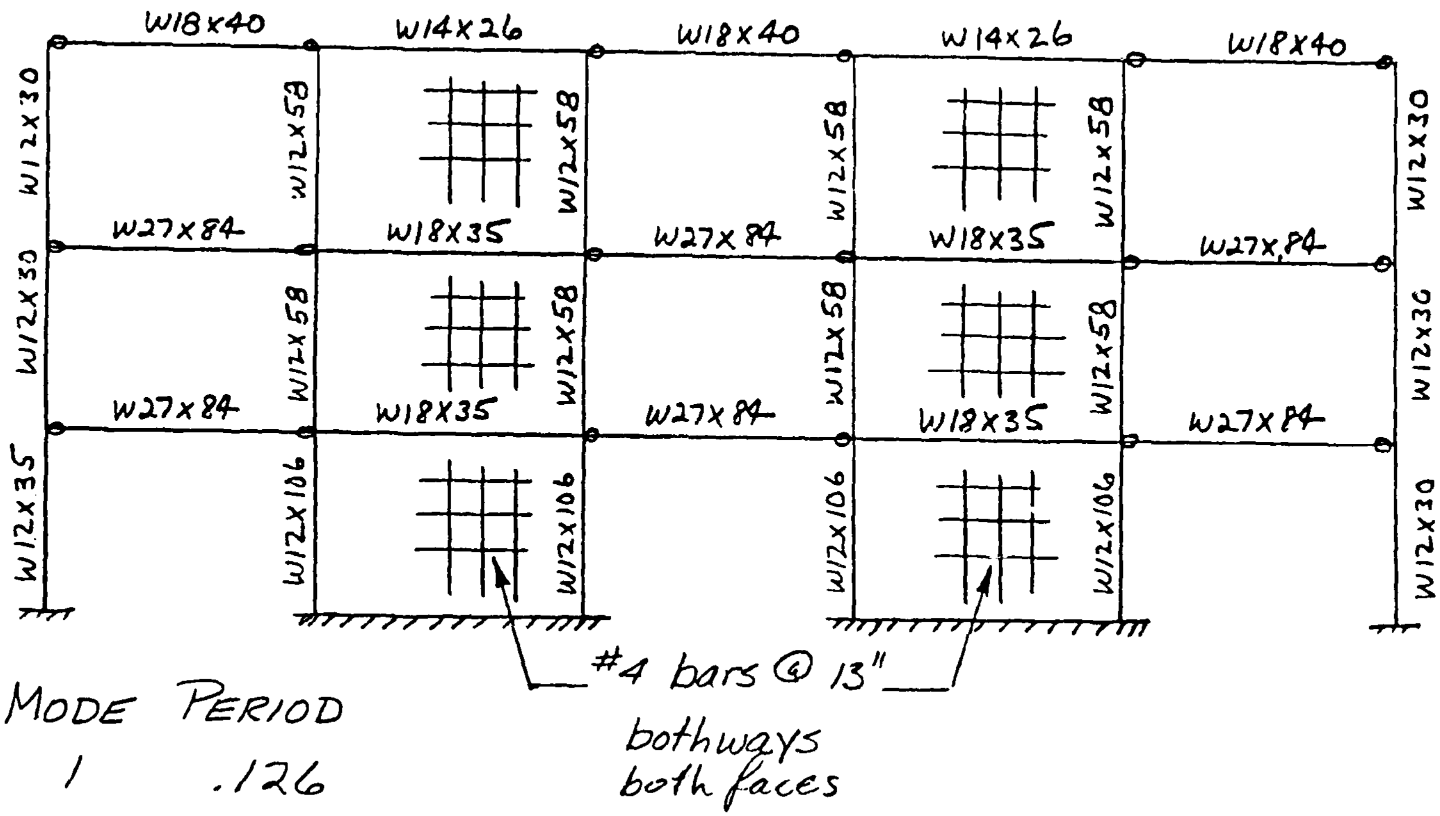


Figure B-14. Shear wall frame, 0.30g base acceleration, 14-inch concrete wall.



MODE	PERIOD
1	.127
2	.044
3	.027

Figure B-15. Shear wall frame, 0.30g base acceleration, 12-inch concrete wall.



MODE	PERIOD
1	.126
2	.044
3	.027

Figure B-16. Shear wall frame, 0.25g base acceleration, 12-inch concrete wall.

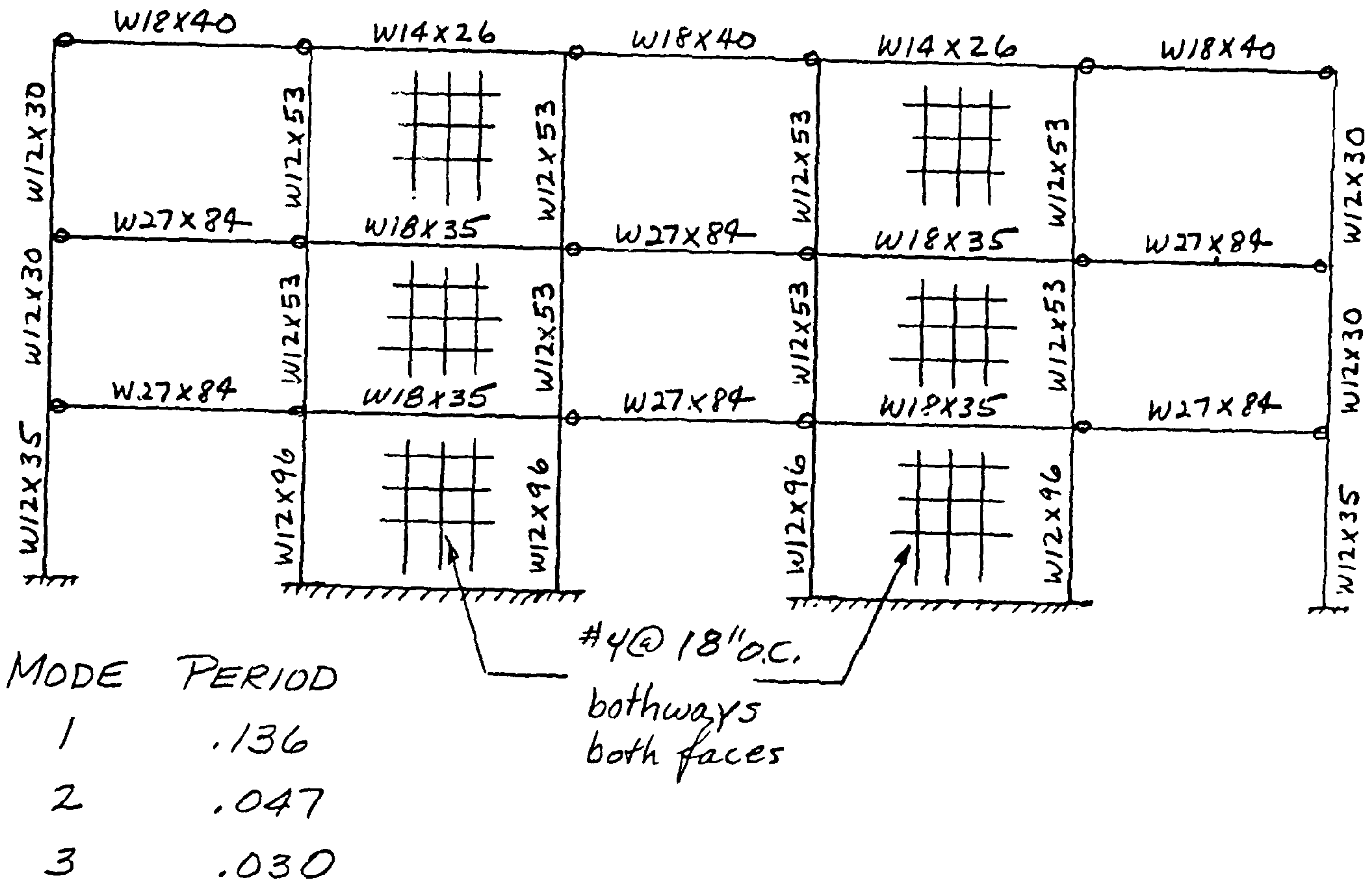
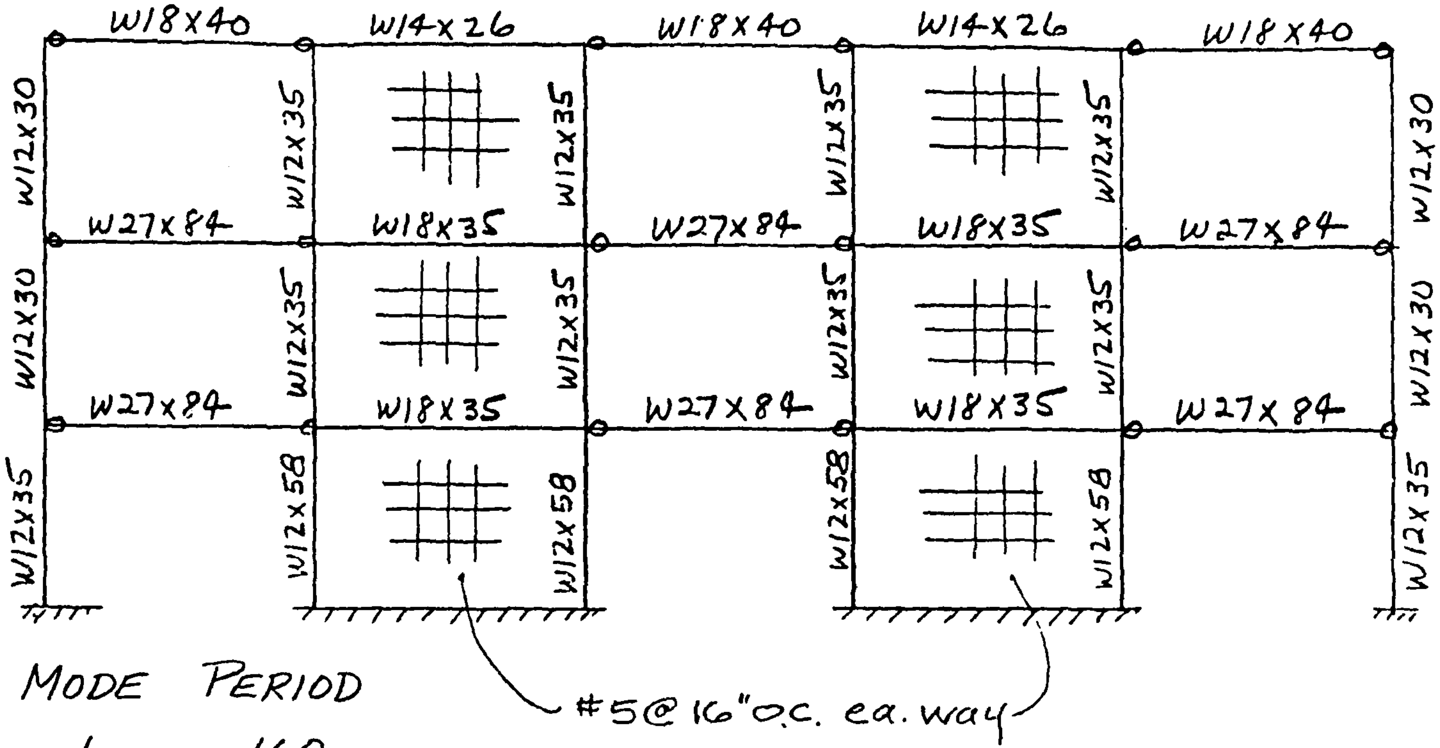
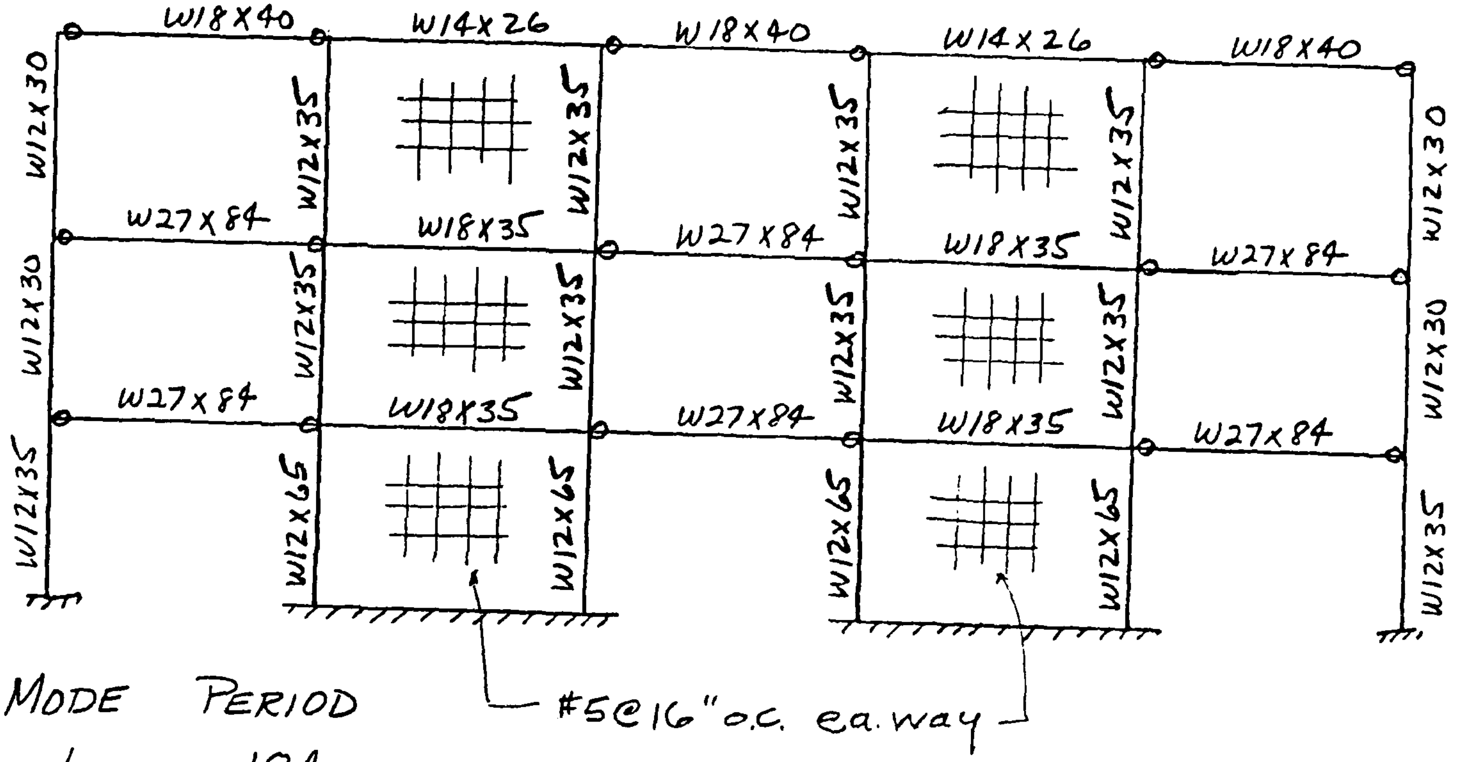


Figure B-17. Shear wall frame, 0.20g base acceleration, 10-inch concrete wall.



MODE	PERIOD
1	.160
2	.056
3	.035

Figure B-18. Shear wall frame, 0.15g base acceleration, 12-inch masonry.



MODE	PERIOD
1	.184
2	.063
3	.039

Figure B-19. Shear wall frame, 0.10g base acceleration, 10-inch masonry.

Appendix C

DAMAGE FUNCTION

DEMAND/CAPACITY RATIO

One of the basic elements in a damage function for a building is the expression of the damage or ductility level as a function of the demand/capacity ratio. The demand/capacity ratio is an expression applied to design acceleration. The basic question is whether a given demand/capacity ratio predicts the same ductility for various levels of design capacity. Murakami and Penzien (Ref C-1) investigated nonlinear response from excitation of single-degree-of-freedom hysteretic models.

Figure C-1 shows the envelope of earthquake records they used; Figure C-2 shows the models. Figures C-3 through C-6 show the linear and nonlinear response for the properties indicated. By use of the data in Figure C-4, the demand/capacity ratio can be plotted for mean ductilities of 1.0 and 5.0 (Figure C-7). Figure C-8, based on Figure C-7, shows the ratio of demand to capacity for a ductility of 5.0 to that for a ductility of 1.0. The ratio is seen to vary with the natural period of the structure. The data indicate that as long as the natural period of a structure remains unchanged, a given demand/capacity ratio will produce the same ductility.

This may be restated as follows: given two elastoplastic single-degree-of-freedom systems with the same mass, stiffness, natural period, and excitation accelerogram shape, the same ductility will be produced in both systems if the ratio of peak acceleration to yield resistance acceleration is maintained.

An alternative formulation has the resistance stiffness and mass of one elastoplastic single-degree-of-freedom system a multiple of the other. The natural periods of both would be the same and, given the same shape excitation with amplitudes, a multiple of the same ductility would result.

It is known (Ref C-2) that damping and period change with ductility as inelastic behavior progresses (Figures C-9 and C-10). However, as noted in Figure C-8, these changes are small and would not influence the ratio of design applied to acceleration for a constant ductility level.

Changes in a structure's design when strengthening a structure primarily influence the stiffness and, to a lesser extent, the mass. Strengthening may be accomplished by substitution of a stronger, or higher yield stress, material of the same stiffness or by an increase in the size of the member, which, of course, would increase the stiffness. These changes in design usually affect the natural period. Figure C-11 shows the stiffness required for elastic ($\mu = 1$) design for nominal design spectrum levels. The data over a range show stiffness required may be approximately taken to be linearly proportional to applied spectral loading.

In actual construction, strengthening will take place by increasing column and shear wall cross sections resulting in some minor increase in mass and a large increase in stiffness. Generally, the optimal design range can be narrowed to approximately the 0.1g to 0.2g range; stiffness might be expected to double and the period to halve over the full range. The ductility can be expected to vary by a factor of 2 over this wide range. Thus, a structure designed for 0.2g with 0.2g applied might have twice the ductility demand of a structure designed for 0.1g with 0.1g applied. Damage estimation relies on estimation of design-to-collapse ratios.

Both the design level and collapse level resistance required depend on the natural period. The ratio of design level (acceleration for $\mu = 1$) to collapse level (acceleration for $\mu = 5$) over a period range of 0.5 to 2.0 seconds varies from 2.25 to 2.60 (Figure C-8). Figure C-12 shows ductility resulting from an application of load acceleration 2.0 times the acceleration for $\mu = 1.0$. Again, between the range of 0.5 and 2.0 seconds, the resulting ductility is fairly constant. This variation is within the accuracy of the data in general and should not have a major effect on the results of the general damage expression formulated. An alternative approach would be to formulate a damage matrix rather than a function.

DAMAGE FUNCTION AND DAMAGE MATRIX

The damage function is an expedient approximate procedure for estimating damage. For existing construction, it is possible to estimate no-damage and collapse-damage levels by scaling response from a single modal analysis. For new construction, it is possible to predict damage by establishing the type of construction and the ratio of elastic to collapse response. These are relatively simple terms requiring minimal analysis. As shown earlier, modal analysis may be an inexact solution to nonlinear response to a random earthquake. Further, changes in natural period affect the damage function.

A further refinement is possible by creating a damage matrix in which the damage condition is given as a function of various design acceleration levels and various nominal* applied acceleration levels.

REFERENCES

C-1. University of California, Earthquake Engineering Research Center. Report No. EERC 75-38: Nonlinear response spectra for probabilistic seismic design and damage assessment of reinforced concrete structures, by M. Murakami and J. Penzien. Berkeley, Calif., Nov 1975.

C-2. National Bureau of Standards. NBS BSS 61: Natural hazards evaluation of existing buildings, by L. Culver et al. Washington, D.C., Jan 1975.

*The term nominal is used to indicate that the acceleration level is based on a spectrum and will contain higher values as a function of frequency.

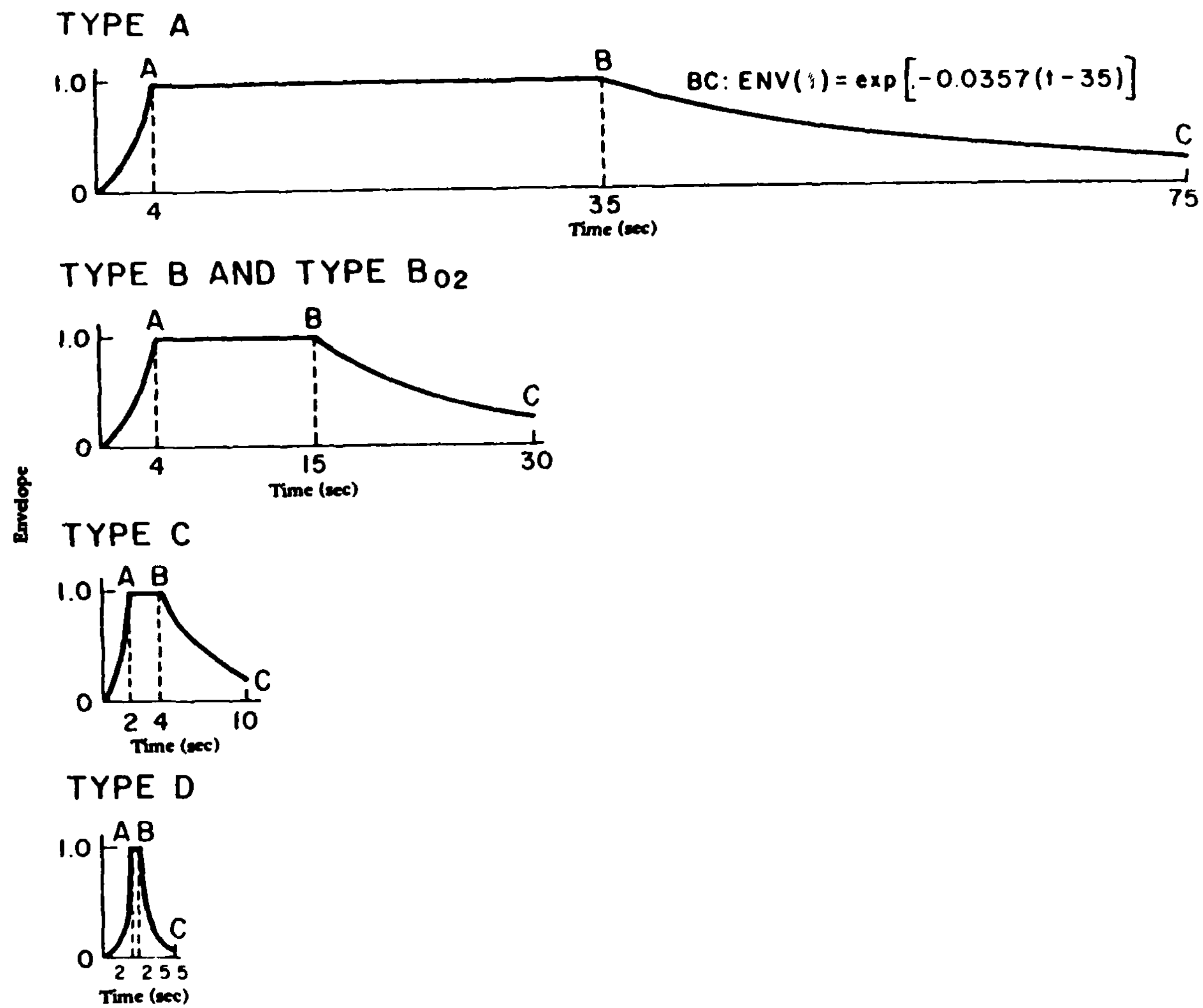
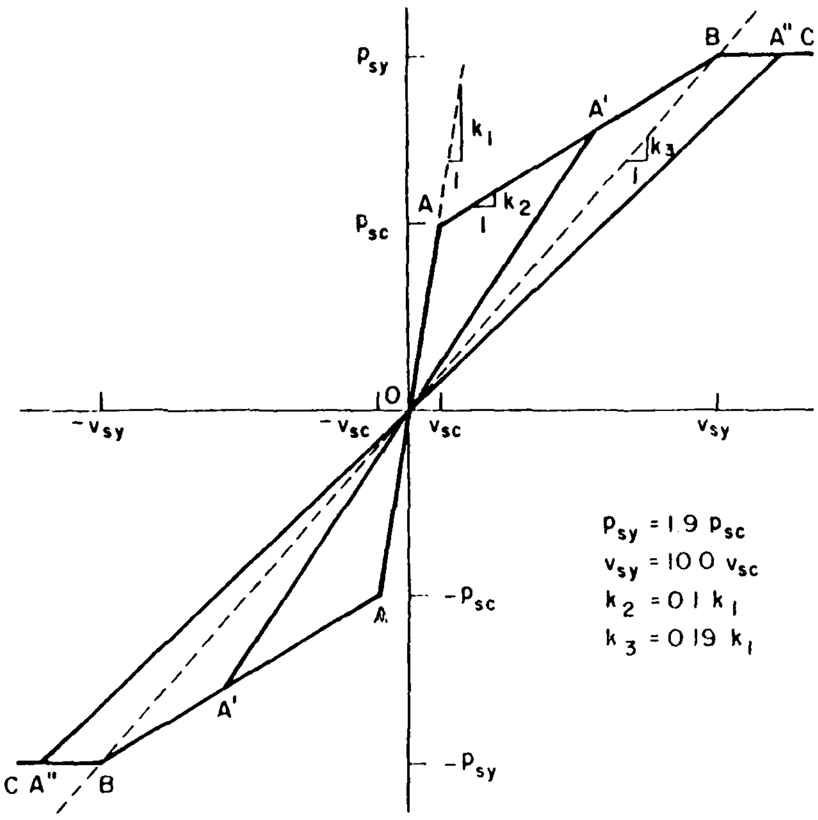
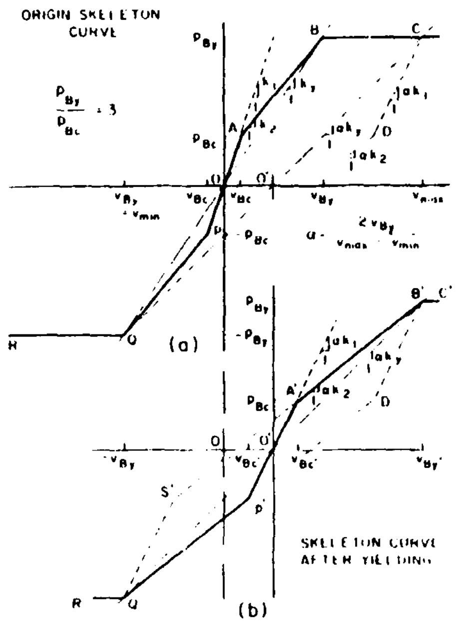


Figure C-1. Time intensity functions (Ref 1).



(a) Origin-oriented hysteretic model.



(b) Tri-linear hysteretic model.

Figure C-2. Structural models (Ref 1).

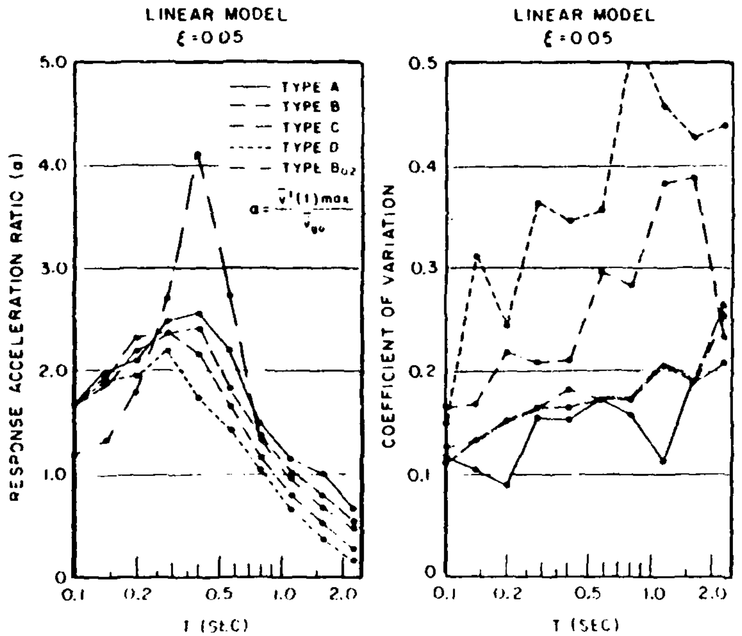


Figure C-3. Response acceleration ratios for linear model (Ref 1).

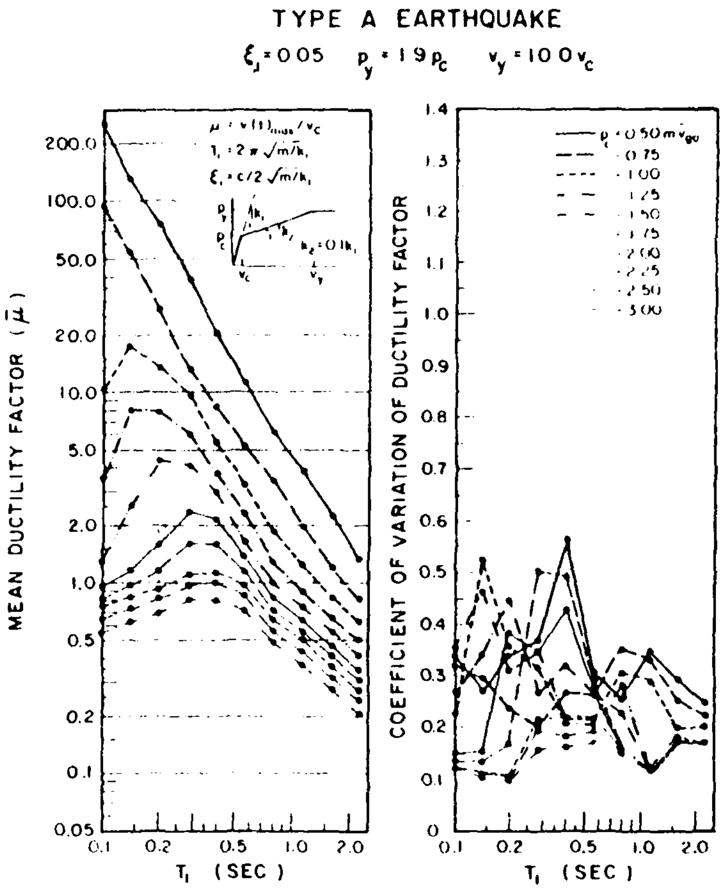


Figure C-4. Mean ductility factors and corresponding coefficients of variation for origin-oriented model having different strength levels for type A earthquake (Ref 1).

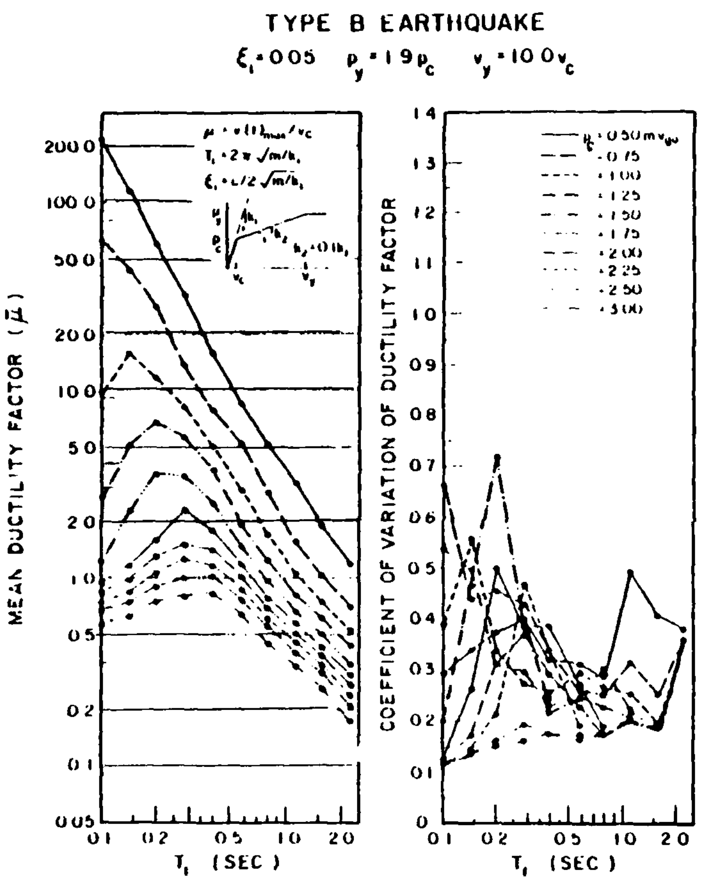


Figure C-5. Mean ductility factors and corresponding coefficients of variation for origin-oriented model having different strength levels for type B earthquake (Ref 1).

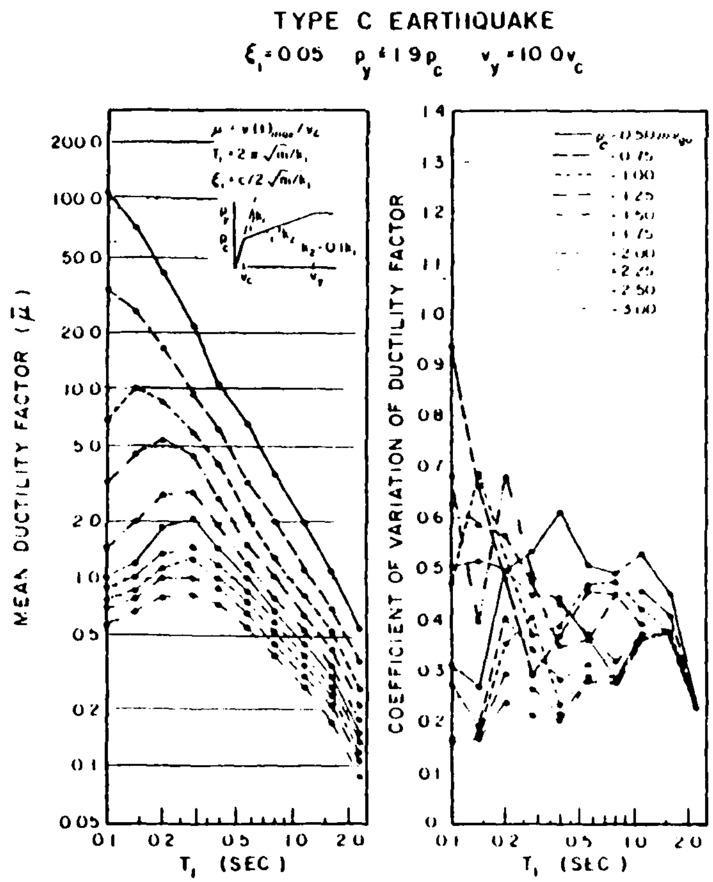


Figure C-6. Mean ductility factors and corresponding coefficients of variation for origin-oriented model having different strength levels for type C earthquake (Ref 1).

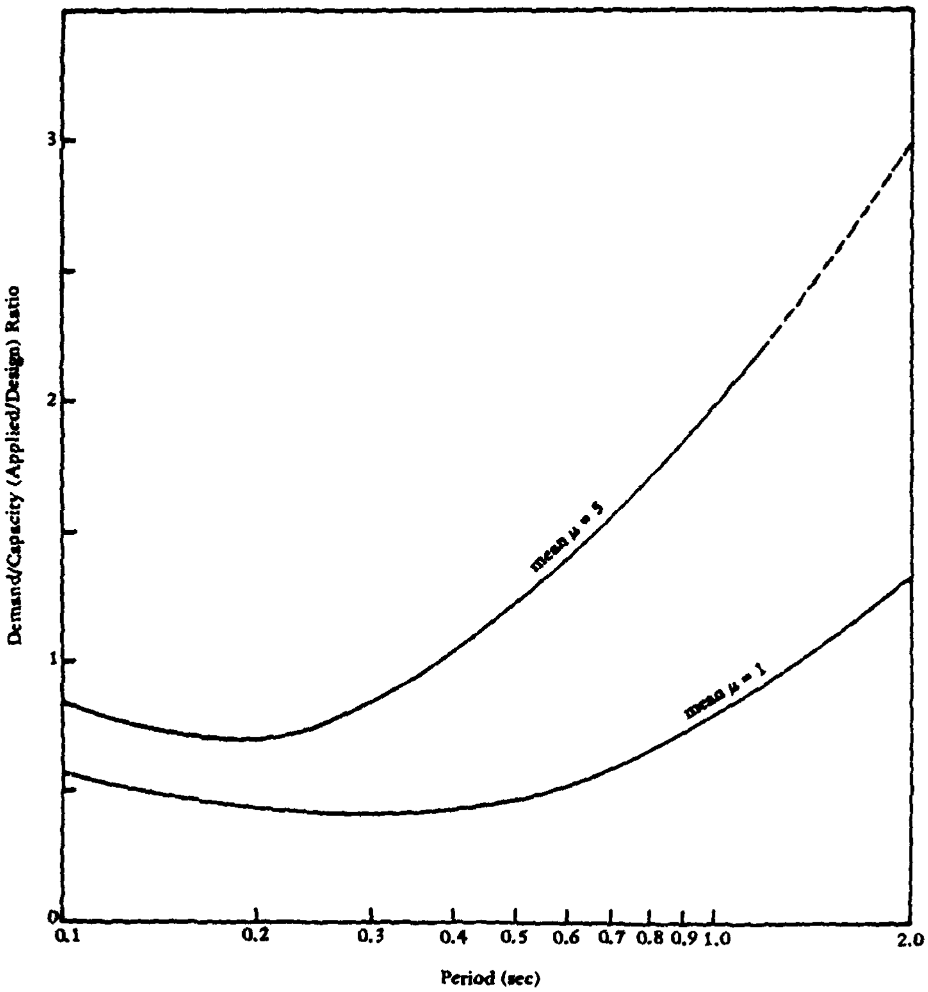


Figure C-7. Mean ductility.

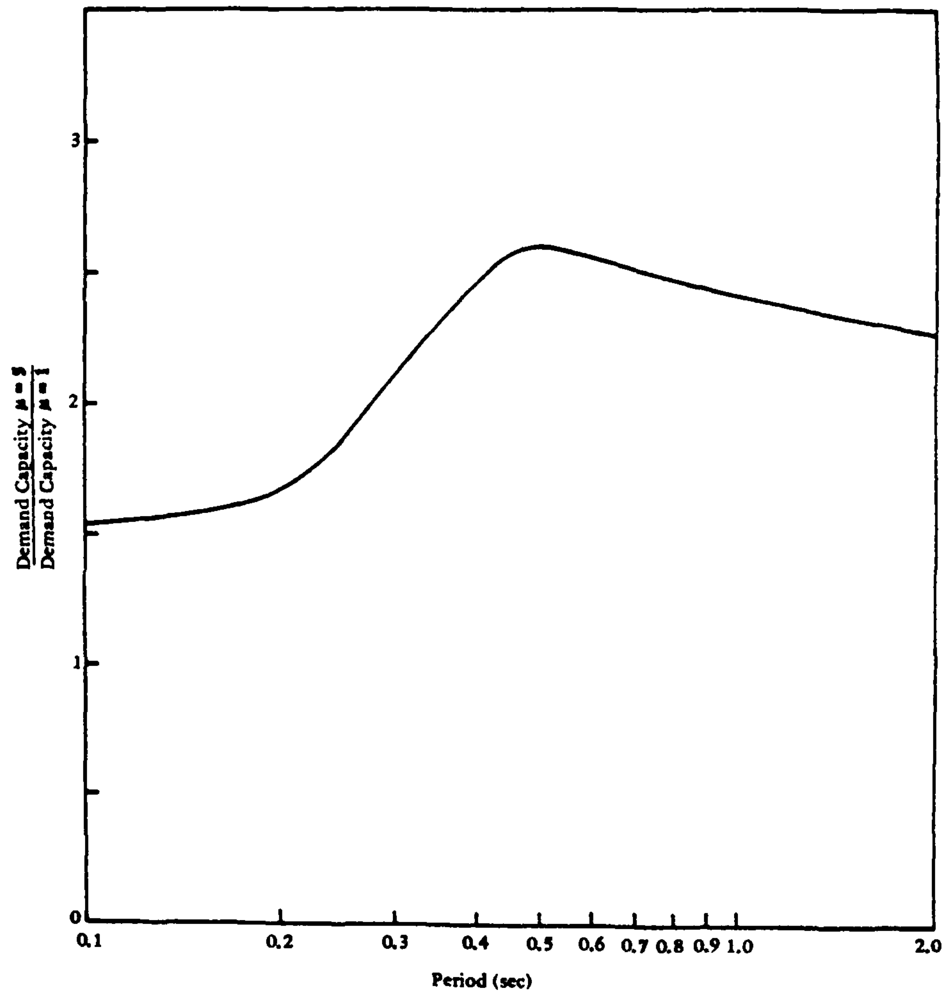


Figure C-8. Ratio demand/capacity $\mu = 5$ to demand/capacity $\mu = 1$.

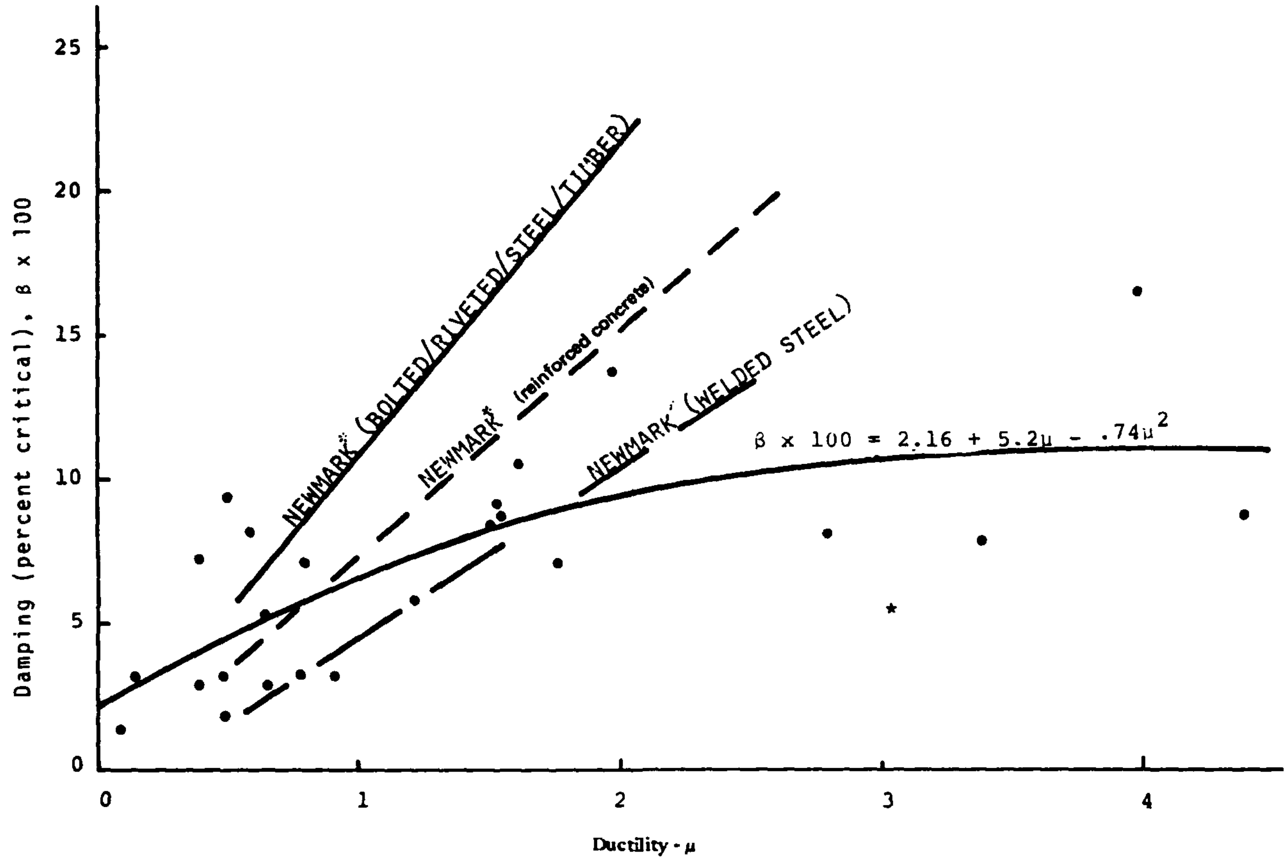


Figure C-9. Variation of damping as a function of ductility (Ref 2).

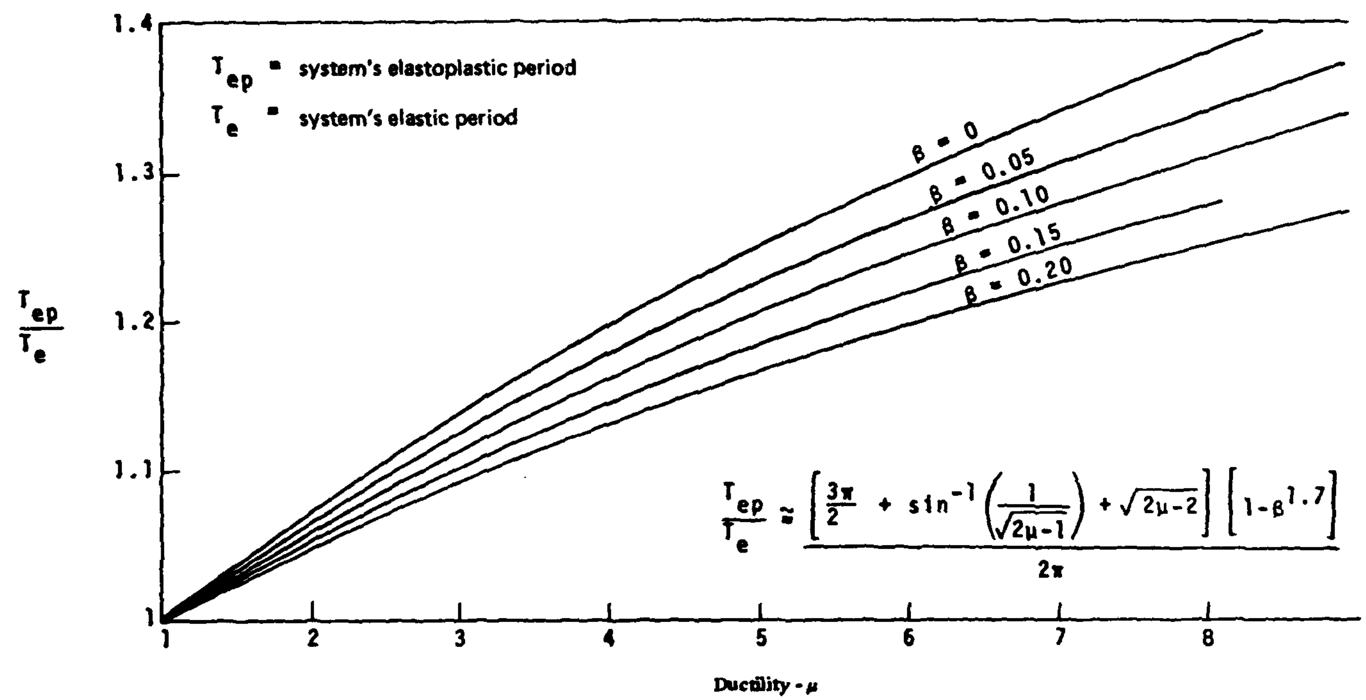


Figure C-10. Ratio of elastoplastic to elastic periods (Ref 2).

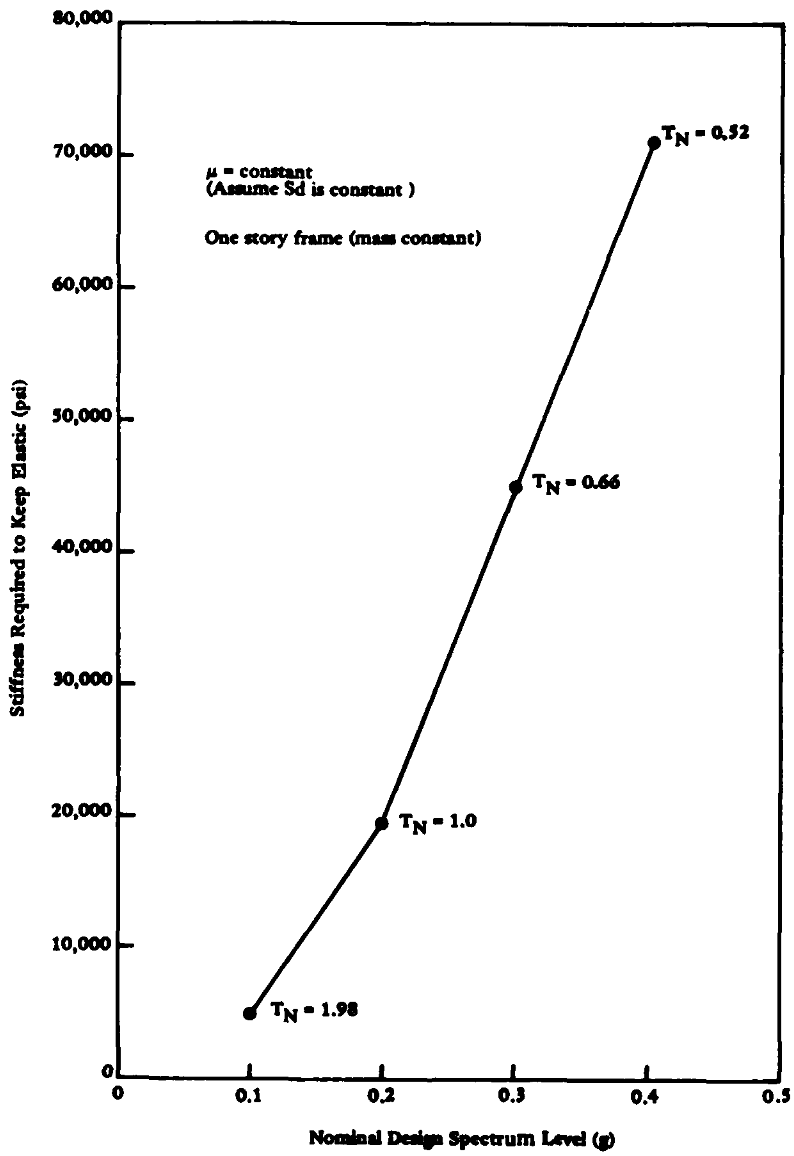


Figure C-11. Stiffness required to remain elastic.

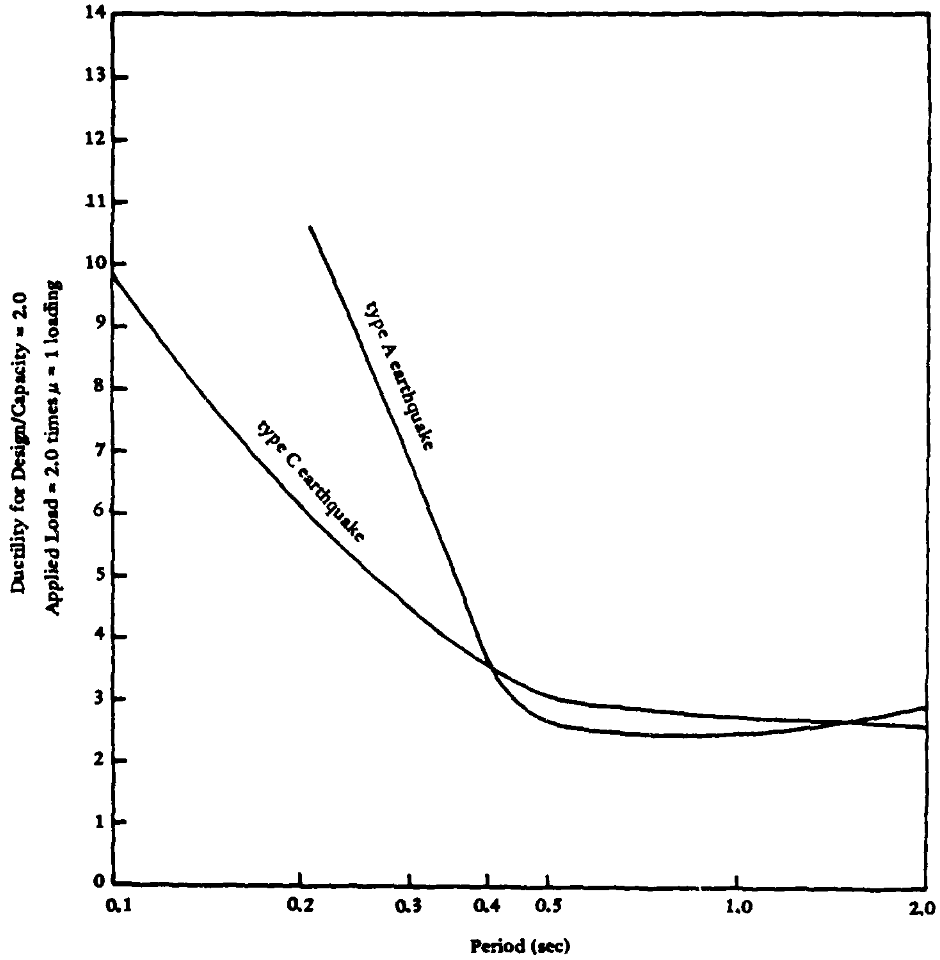


Figure C-12. Ductility variation with period for load 2.0 times $\mu = 1$ load.

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